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HIGHWAY RESEARCH RECORD

Number 374

Soil: Compaction, Classification, and Laterites

7 Reports

HIGHWAY RESEARCH BOARD

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Subject Areas

33	Construction
61	Exploration-Classification (Soils)
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CONTENTS

FOREWORD

This RECORD is composed of papers concerning different aspects of roadway design, construction, and performance. For example, Youd describes a method of obtaining the maximum dènsity of sand by repeated strain in simple shear. This is useful in studies and control of relative density, especially where the earth structure involves earthquake or landslide design. In contrast, McDowell and Smith develop a compaction control method for use in construction control using a gyratory specimen to obtain the density ratio in relation to a field control test hole. They cite many advantages, including rapidity of test results and more direct relation to material being tested. Williams and Marais present a solution to a problem peculiar to certain areas, that of collapsing soils, by recommending the use of an impact roller. They report good results with their method. Yandell describes the effects of repeated loading on a roadway section. His analysis shows that the road material flows forward against the direction of traffic movement in a complicated but predictable manner, which may lead to roadway corrugations.

Witczak, Lovell, and Yoder studied the general potential for poor soil support based on the occurrence of organic and clayey deposits. They developed a national soil textural map and rated the severity of the problem in 97 physiographic sections in the 48 conterminous states.

Gidigasu describes the development of parameters for classification of fine-grained laterites. These are affected by degree of weathering, morphological characteristics, and chemical and mineralogical composition as well as by environmental conditions. Townsend, Manke, and Parcher discuss the influence of sesquioxides on lateritic soil properties and present data regarding stabilization of these soils.

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MAXIMUM DENSITY OF SAND BY REPEATED STRAINING IN SIMPLE SHEAR

T. Leslie Youd, U. S. Geological Survey, Menlo Park, California

A new method is described for determining the maximum densities of granular soils in the laboratory. The method consists of applying many repeated cycles of shear strain (40 to 60 cycles/min) to samples mounted in a simple shear apparatus. Maximum densities produced by this method exceeded maximum densities reported in the literature or obtained from standard ASTM vibratory procedures by 2 to 6 pcf (pounds per cubic foot) for several clean sands of varying gradation. For a silty sand (crushed basalt) having 15 percent of its particles finer than a No. 200 sieve, the maximum density obtained by the new method was 11 pcf greater than that measured by the standard ASTM vibratory procedure (D 2049-69) and 4 pcf greater than that measured by the ASTM modified Proctor compaction test (D 1557-70). The new method gave only a slightly greater density than a modified Proctor compaction test on a silty sand having 32 percent of its particles finer than a No. 200 sieve. The results show that the maximum densities obtainable by repeated cycles of shear strain are consistently higher than those obtainable by standard ASTM procedures for clean and moderately silty sands.

•RELATIVE density is an important measure of the density state of a granular soil. Its use has been hampered in part by the lack of an adequate laboratory technique for obtaining maximum density. For relatively clean sands, greater densities have been obtained in the laboratory from vibratory tests than from other methods. However, vibratory densities are not necessarily absolute maximum densities (5). Moreover, they are valid only for relatively clean sands. Other methods, such as impact techniques, must be used for siltier sands.

Recent studies have shown that granular materials can be compacted appreciably by repeated cycles of shear strain (1, 6, 7). The purpose of this study was to determine whether the repeated shear straining process would have advantages over vibrational techniques for determining maximum densities of granular soils in the laboratory, with respect to both the densities obtained and the range of gradations for which the technique is valid.

APPARATUS AND PROCEDURE

A Norwegian Geotechnical Institute type of simple shear apparatus, modified after the manner described by Thiers and Seed (8), was used to apply repeated shear strains to samples of granular soils (Fig. 1). The sample was confined laterally by a wire-reinforced rubber membrane and constrained vertically between end caps bonded with sandpaper. The top cap was free to displace vertically but was prevented from rotating or moving horizontally. The bottom cap could be displaced horizontally during a test but was restricted from any other mode of movement. Repeated shear strains were applied to the sample by cyclically displacing the base. These displacements were actuated by an eccentric cam, which was in turn driven by a variable-speed motor.

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Vertical load was applied through the top cap either by increasing the cap load with air pressure acting on an air cylinder or by relieving it with a counterbalancing spring.

Five different sands were tested (Fig. 2): two standard gradations of Ottawa sand (ASTM C-109 and ASTM C-190), a beach sand from Rodeo Cove, California, a mechanically crushed basalt, and a dune sand from the Mojave Desert, California.

Sand samples to be tested were poured into the membrane, which had been pre-mounted on the bottom cap (all samples were tested in an air-dry condition). This cap was tapped lightly to compact the sand to a moderate density. The top cap was placed on the sample and the assembled unit placed in the shear device. The desired normal load was applied, and the sample was subjected to repeated cycles of shear strain until a maximum density was attained (at least 10,000 strain cycles were applied to each sample). Shear displacements were between 0.040 and 0.080 in. (peak to peak) unless otherwise specified, and frequencies ranged from 40 to 60 cycles per minute (cpm). Preliminary tests showed that neither saturation nor frequency (up to 120 cpm) significantly influenced the maximum densities obtained.

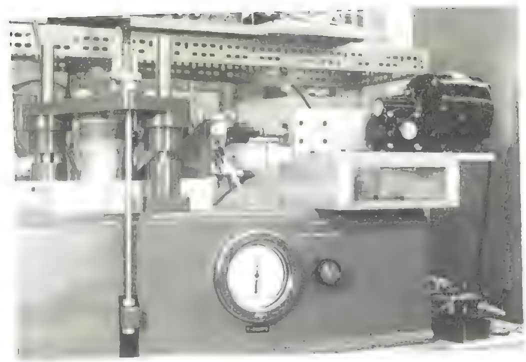

Figure 1. Simple shear apparatus with sample mounted and ready for testing.

The sample height and diameter (average of several measurements corrected for membrane thickness) were measured to the nearest 0.001 in. before and after each test. The sample height and the shear displacement were monitored during the test with

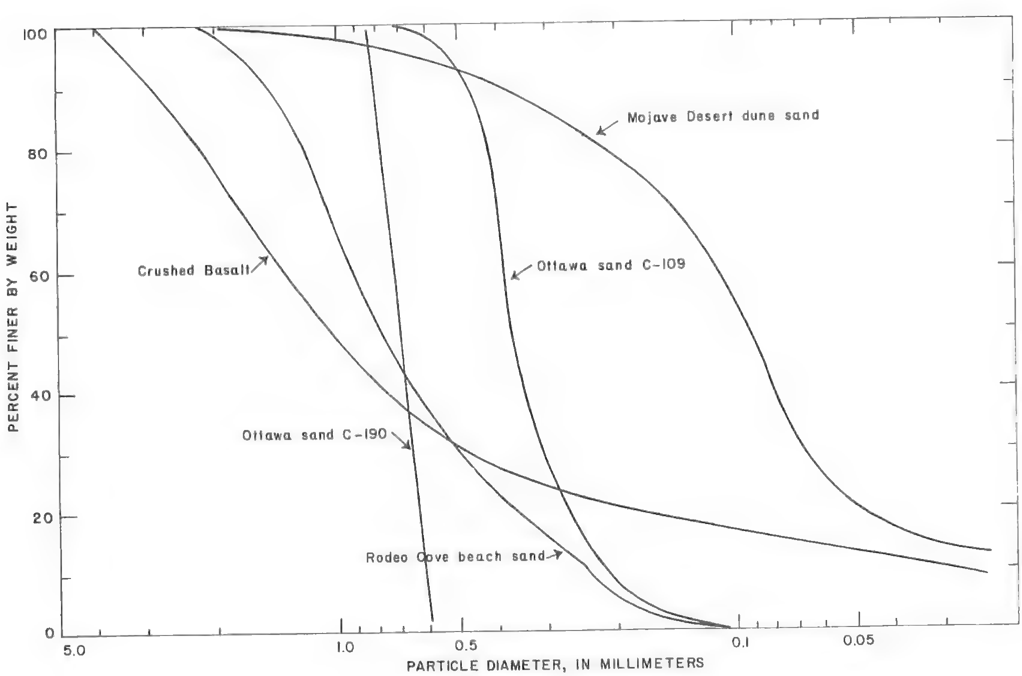

Figure 2. Particle-size distributions of soils used in densification tests.

LVDTs and a multichannel oscillographic recorder. The heights ranged from 0.70 to 0.90 in., the diameter ranged from 3.07 to 3.14 in., and the sample weight (weighed to the nearest 0.1 gram) ranged from 190 to 210 grams. From these measurements the dry densities were calculable to an accuracy of ±0.5 pcf (pound per cubic foot).

RESULTS

The curves in Figures 3 and 4 show that the density of the C-109 Ottawa sand samples increased with each cycle of shear strain until a well-defined maximum density was reached that is independent of shear-strain amplitude and vertical stress for stresses above about 1,000 psf (pounds per square foot). The other sands reacted in a similar manner (Fig. 5) except for the Mojave Desert dune sand, which showed an additional increase beyond the 2,000-psf level. This increase may have been due to crushing of particles (Table 1). From sieve analyses on this sand, it was found that up to 3.5 percent more particles passed through sieves in a nest after testing at 4,000 psf vertical stress than before, whereas only up to 1.7 percent more particles passed through the sieves following a similar 2,000-psf test. That crushing of particles in the other sands was relatively minor is indicated by the data in the table.

TABLE 1

CRUSHING OF PARTICLE DURING 10,000 SHEAR-STRAIN CYCLES AS REVEALED BY PARTICLE SIZE ANALYSIS ON SAMPLES BEFORE AND AFTER STRAINING

Sand	Maximum Increase of Particles Passing Any Sieve (percent)	
	2,000-psf Test	4,000-psf Test
C-190 Ottawa sand	—	0.3
C-109 Ottawa sand	—	0.3
Rodeo Cove beach sand	—	0.2
Crushed basalt	—	1.2
Mojave Desert dune sand	1.7	3.5

Table 2 gives the densities obtained in the simple shear apparatus compared with (a) published data on Ottawa sand C-190, (b) the standard ASTM vibratory tests (D 2049-

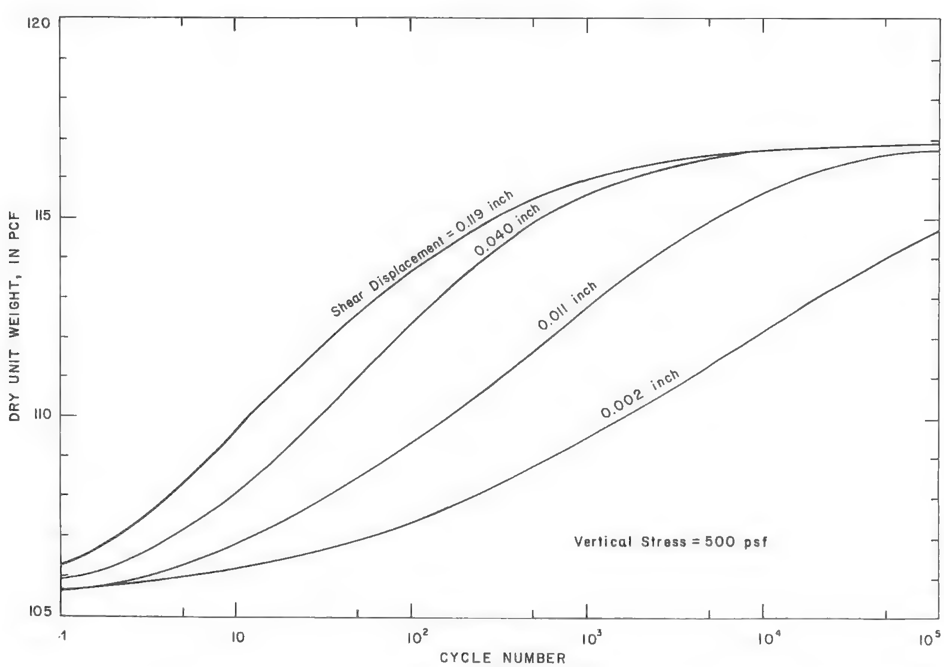

Figure 3. Densification histories of C-109 sand samples tested at different cyclic shear displacements.

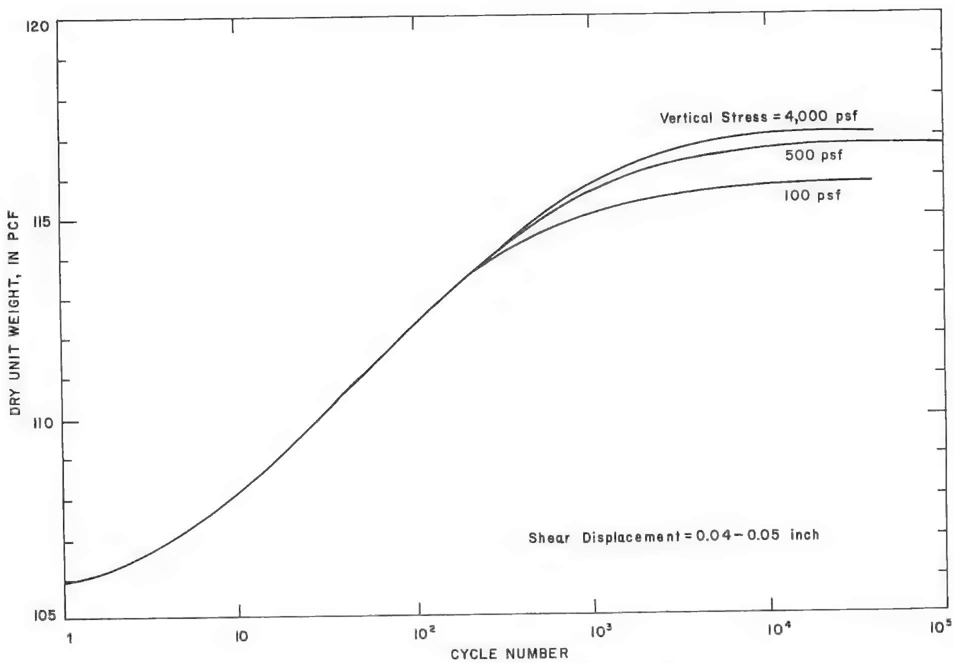

Figure 4. Densification histories of C-109 sand samples tested at different vertical loads.

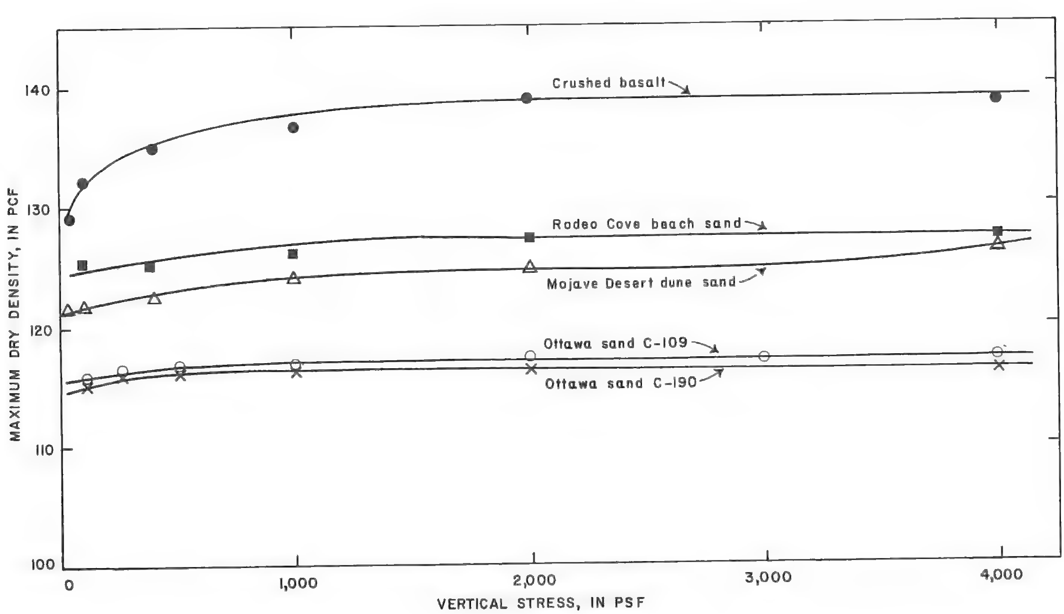

Figure 5. Maximum densities of five sands determined by repeated straining method.

TABLE 2

COMPARISON OF SHEAR-STRAIN TEST MAXIMUM DENSITIES WITH THOSE FROM STANDARD TESTS AND PUBLISHED DATA

Sand (1)	Percent Passing No. 200 Sieve (2)	Standard Test or Published Densities: Minimum Density (pcf) (3)	Maximum Density (pcf) (4)	Test Method or Source (5)	Repeated Shear-Strain Densities: Maximum Density[c] (pcf) (6)	Relative Density (percent) (7)
Ottawa sand C-190	0	92	110	Hough (4)	116.3	126
		92[a]	114[a]	Drnevich and Richart (3)		108
Ottawa sand C-109	0	94.4	111.2	ASTM D 2049-69[b]	117.1	128
Rodeo Cove sand	0.3	106.7	125.0	ASTM D 2049-69[b]	127.3	110
Crushed basalt	14.8	97.9	127.8	ASTM D 2049-69[b]	138.7	125
			134.5	ASTM D 1557-70		
Mojave Desert sand	32.5		123.9	ASTM D 1557-70	124.6	

[a]Assuming a specific gravity of 2.65.
[b]Tests conducted by a commercial testing laboratory.
[c]Compacted under a vertical stress of 2,000 psf.

69) for Ottawa sand C-109, Rodeo Cove sand, and crushed basalt, and (c) the standard ASTM modified Proctor test (D 1557-70) for the crushed basalt and Mojave Desert sand. The relative densities in Column 7 were calculated from the maximum density obtained by repeated shear strain (column 6) together with the maximum and minimum densities obtained by standard methods (columns 3 and 4). The results show that, in every case, the maximum densities achieved by repeated shear straining were appreciably greater than those determined with standard methods.

Table 2 further shows that the cyclic shear straining method is applicable to a wider range of soils than are standard vibratory tests. ASTM Designation D 2049-69 (2) is restricted to soils containing less than 12 percent particles finer than a No. 200 sieve, and the U. S. Bureau of Reclamation test designation E-12 (9) is limited to free-draining soils. The cyclic shear-strain technique yielded appreciably greater densities than either the vibratory or impact methods on the crushed basalt (14.8 percent passing the No. 200 sieve), and a slightly greater density than that obtained with the modified Proctor method for the Mojave Desert sand (32.5 percent passing the No. 200 sieve).

NEED FOR FURTHER DEVELOPMENT

Simple shear devices of the type described here are found in only a few research laboratories. Even if they were generally available, their usefulness for obtaining the maximum density of sands would be limited in part by the small sample size (about 7 cu in.) and the slowness of the test (about 3 hours of cyclic shearing is required for each test). Another disadvantage of this equipment is its cost (about $3,000). It is suggested that development of a less costly apparatus with a larger shear box and the capability of operating at much higher frequencies is required before the repeated shear-strain technique can be routinely applied in the laboratory.

CONCLUSIONS

1. A well-defined maximum density occurs during repeated shear straining of granular soils.
2. This maximum density exceeds that obtained with existing standard procedures.
3. Repeated straining in simple shear appears to be a valid technique for obtaining the maximum density of silty as well as clean sands.

ACKNOWLEDGMENTS

The assistance of Sam Shaler, who performed the densification tests, is gratefully acknowledged. Publication has been authorized by the Director, U. S. Geological Survey.

REFERENCES

1. Anderson, G. F. An Earthquake Hazard Study of a Dune Sand Site in San Francisco, California. Master's thesis, San Jose State College, 1969.
2. 1970 Annual Book of ASTM Standards, Part 11. American Society for Testing and Materials, Philadelphia, 1970.
3. Drnevich, V. P., and Richart, F. E. Dynamic Prestraining of Dry Sand. Jour. Soil Mechanics and Foundations Div., Proc. ASCE, Vol. 96, No. SM2, 1970, pp. 453-469.
4. Hough, B. K. Basic Soils Engineering. Ronald Press Co., New York, 1957, 513 pp.
5. Johnson, A. W., and Sallberg, J. R. Factors Influencing Compaction Test Results. HRB Bull. 319, 1962, 148 pp.
6. Richart, F. E., Hall, J. R., and Woods, R. D. Vibrations of Soils and Foundations. Prentice-Hall, Inc., Englewood Cliffs, N. J., 1970, 414 pp.
7. Silver, M. L., and Seed, H. B. The Behavior of Sands Under Seismic Loading Conditions. Univ. of California, Berkeley, Report No. EERC 69-16, 1969, 64 pp.
8. Thiers, G. R., and Seed, H. B. Cyclic Stress-Strain Characteristics of Clay. Jour. Soil Mechanics and Foundations Div., Proc. ASCE, Vol. 94, No. SM2, 1968, pp. 555-569.
9. U. S. Bureau of Reclamation. Earth Manual. U. S. Gov. Printing Office, 1968, 783 pp.

COMPACTION CONTROL BY RATIO OF VOLUME OF GYRATORY SPECIMENS TO VOLUME OF TEST HOLES

Chester McDowell and Avery W. Smith, Texas Highway Department

Eighteen soil materials varying from subgrade soils to flexible base materials were investigated for the relationship between the commonly used Texas compaction ratio density and the density obtained by certain gyratory compaction procedures. The degradation of three other soft crushed limestone samples caused by use of the gyratory procedure was investigated. The data obtained indicate that a method has been established whereby gyratory densities (when modified with a compaction K factor) can be related to the compaction ratio densities that are commonly specified in Texas. The method provides for testing the same material that is obtained in digging test holes, thereby eliminating the necessity for performing preliminary moisture-density curves and also the "guessing game" for identifying the proper moisture-density curve for use. For most flexible base materials, the method should be fairly rapid in that moisture content determinations can be eliminated because dry weight is a constant and cancels out of the calculations. Results indicate that a rather large amount of degradation does occur but its presence does not cause the shape or magnitude of moisture-density relationship curves to change.

•MUCH has been said during the past 35 years about density control of soil materials, although there is no assurance that the moisture-density curves tested in the laboratory are applicable to the material being measured in the roadway. The efficiency of methods has depended heavily on choosing the correct moisture-density curve for use. The chances of being able to make a good selection are slim unless the material in question is remarkably uniform in many respects. Another problem has been the determination of the degree of density needed to compact our wide variety of soils to achieve the most desirable conditions within the realm of economic reason.

No well-informed person would claim that he can specify a single density control test result that would be ideal for a given soil in all positions in various types of soil structures. This is because loading conditions, strength requirements, degree of permeability, and volume change potential will all vary with each type of facility. If given enough time and funds, a soils research engineering organization could find the proper answers, but no governmental organization has been able to afford such expenditures for road and street work. Even if the ideal moisture-density condition were known, the soils engineer would often have to make difficult decisions. He may be faced with high costs of altering extensive layers of natural soil deposits in order to comply with some "ideal" requirement of moisture and density. There are cases where the expenditure required to meet "ideal" requiremements are not always necessary or justifiable. It is in cases of this type that a soils engineer can be of great assistance. As an example, he may find that the soil is denser than required and is nonswelling; therefore, it would be foolish to loosen the soil. On the other hand, if the soil is looser than required, it may be that redesigning the structure would be more economical than compacting a valley full of natural partially compacted deposits. It is even more difficult to decide on the most practical and economical steps to follow in building over clay soils. Many

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times the swelling soil conditions are such that layers need to be ponded or ripped up and treated with water or lime before replacing. At the same time pavements should be designed such that moisture fluctuation is minimized. Although knowledge of how to minimize volume change exists, there are some light traffic roads where it may not always be economical to execute the work in the manner desired. In other words, it may be more economical to rebuild the base and surfacing in stages rather than to spend enormous sums of money initially. There are many other facilities to be constructed on clays that cannot be approached from the stage construction standpoint, and therefore their foundations may require redesign to be the most economical. Knowledge of "potential vertical rise" (4, 5) is necessary to arrive at practical conclusions pertaining to use of stage construction.

If strength is to be considered in design, then some generalized value of density control is necessary for road and street construction. Such a generalized method known as "compaction ratio" (1, 2) has been used sucessfully by the Texas Highway Department for many years. The method has the following advantages:

1. The method is based on the construction and performance of numerous highways that have been in service many years;
2. The method has wide application to soils varying from clays, silts, and sands to flexible base materials;
3. The method was founded on the premise that highest practical strengths within reason would be obtained and that overdensification and drying out of swelling clays would not be permitted; and
4. The degree to which soil materials are compacted is based on the compactibility of the materials involved.

Construction moves at such a pace that quick, accurate answers are necessary. In order to obtain the maximum benefit from the strength of soil materials, it is necessary that the answers be accurate in the sense that they represent the materials in question. In the past, we were never certain that the appropriate moisture-density curve was available for the material excavated from the density test hole. In 1966 we stated (3), "To offer considerable relief from the problems created by attempting to control density of nonuniform and erratic materials, we need to develop a density test, which can be run on the material excavated from the density holes."

We now propose to test the material from the density test hole in the gyratory press (Figs. 1 and 2) in such a manner that

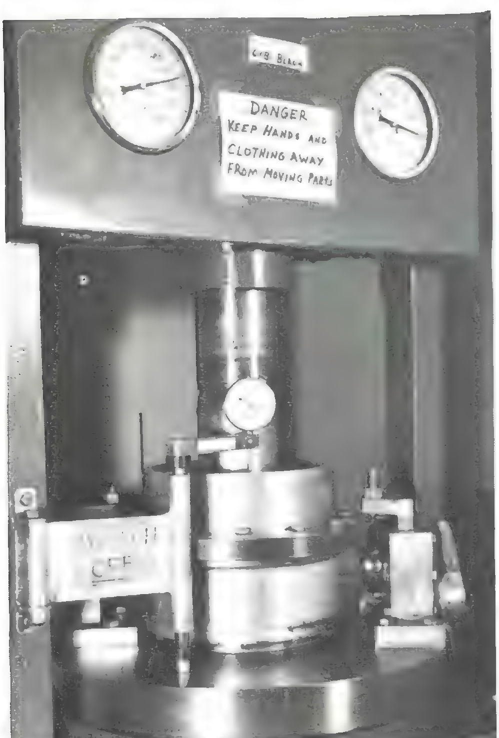

Figure 1. Measuring height of specimen.

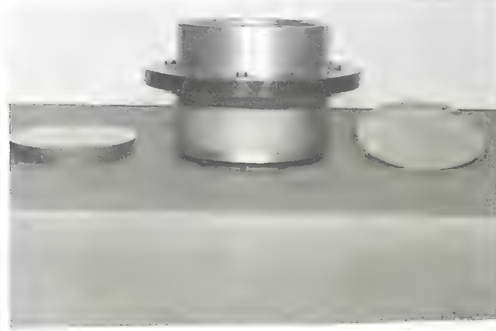
Figure 2. Compaction mold and base plate.

TABLE 1

RELATIONSHIP OF D_G, D_A, AND D_{GW} OF FLEXIBLE BASE, SUBBASE, AND SUBGRADE MATERIALS ON 7-INCH BY 3-INCH GYRATORY SOIL COMPACTED SPECIMEN

Laboratory Number	D_G, Maximum Dry Density	D_A, Actual	D_{GW}, Maximum Wet Density	$\frac{D_G}{D_A}$
61-154-R	153.2	146.7	159.2	1.04
62-69-R	145.3	140.6	155.2	1.03
58-9-R	122.5	110.6	138.8	1.11
66-192-R	115.7	93.9	132.9	1.23
39-11-MR	113.5	92.2	132.7	1.23
62-5-R	125.8	117.2	140.0	1.07
66-184-R	122.7	106.8	138.7	1.15
64-459-R	145.1	139.4	152.9	1.04
63-383-R	139.7	133.4	149.9	1.05
64-413-R	142.1	136.5	150.5	1.04
63-233-R	151.5	145.4	157.3	1.04
58-232-R	142.4	136.1	152.1	1.05
65-67-R	142.0	137.1	151.2	1.04
65-100-R	159.2	150.9	164.6	1.06
63-282-R	143.4	135.1	152.4	1.06
67-194-R	148.3	139.9	155.1	1.06
69-208-R	124.4	113.3	139.3	1.10
69-209-R	124.2	115.9	138.4	1.07

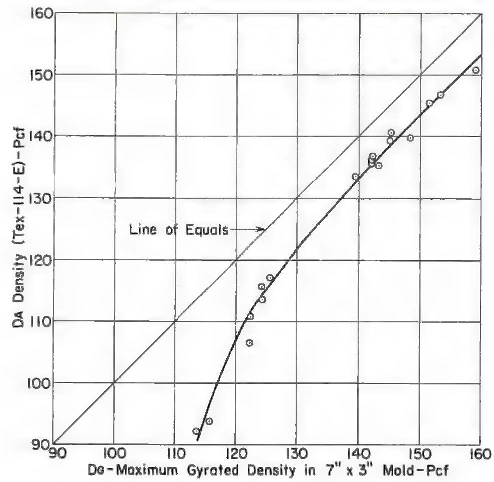

Figure 3. Relationship of compaction ratio densities to gyrated dry densities.

percent compaction ratio density can be determined by use of the ratio of the minimum gyratory volume (times a factor) to the density test hole volume. The relationships and calculations show that the proposed procedure is such that dry weight of material is constant and cancels out of the equation. The dry density can be obtained by drying the sample, but it is not needed for determination of percent compaction ratio density. An investigation of 18 widely varying soil materials showed a relationship between compaction ratio and gyrated densities (Table 1 and Fig. 3).

After deriving the percent density formula, it became necessary to determine realistic values for the term D_G/D_A, or K value, before results could be implemented. Figure 4 shows the relationships between the gyrated wet densities D_{GW} of these materials to their respective D_G/D_A ratios. (Table 2 gives gradation and soil constants.) The idea of using K values in implementing other compaction procedures to embrace the use of compaction ratio densities could have broad application, but a new set of K values for each additional compaction system would have to be determined.

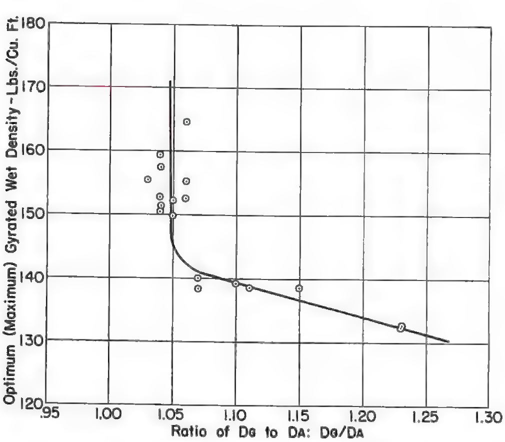

Figure 4. Optimum gyrated wet density versus D_G/D_A (K factor).

The following is how we determined percent compaction ratio density by comparing volumes of gyratory specimen and field density holes.

Where the material is a fixed amount of dry soil material taken from a density test hole and

DW = dry weight of soil,
V_{dh} = volume of density test hole,
V_{DA} = volume of same material would be at D_A density,
D_A = compaction ratio dry density,
D_G = optimum dry density of gyrated specimen (maximum density),
D_{GW} = optimum wet density of gyrated specimen (maximum density), and
V_G = optimum volume of gyrated specimen (minium volume).

TABLE 2

PHYSICAL CHARACTERISTICS OF GYRATED SOILS MATERIALS

Laboratory Number	Soil Constants							Percent Retained on Square Mesh Sieves												Grain Diameter (millimeters)			Specific Gravity	Material
	LL	PI	SL	LS	SR	Soil Binder	WBM Percent Loss	1¾	1¼	⅞	⅝	⅜	4	10	20	40	60	100	200	0.05	0.005	0.001		
61-154-R	20	6	13	4.3	1.97	19		0	3	22	30	33	45	56	73	81	84	85	86	87	94	97	2.71	Crushed stone flexible base
62-69-R	15	2	13	1.8	1.85	47					0	5	33	49	50	53	59	69	82	85	95	96	2.59	Iron ore subbase
58-9-R	34	20	18	8.0	1.75	94									0	6	7	34	50	51	58	64	2.69	Subgrade soil-clay-sand
66-192-R	55	32	16	16.2	1.80	99									0	1	1	2	3	4	30	62	2.66	Subgrade soil-clay
39-11-MR	61	41	11	20.0	1.93	100										0	1	4	8	9	45	59	2.71	Subgrade soil-clay
62-5-R	26	4	22	3.7	1.70	100											0	3	27	43	84	90	2.67	Subgrade soil-sand-clay
66-184-R	42	24	14	14.0	1.89	98									0	2	5	15	28	32	55	69	2.73	Subgrade soil-clay
64-459-R	19	5	14	3.7	1.93	21	32	0	6	20	33	46	59	68	75	79	81	83	85	87	95	98	2.72	Crushed stone flexible base
63-383-R	28	13	16	6.0	1.88	34	45	0	8	21	32	42	54	58	61	66	69	72	75	77	87	94	2.73	Crushed stone flexible base
64-413-R	32	6	24	4.0	1.58	21	34	0	7	24	35	45	56	66	75	79	82	85	89	91	97	98	2.62	Crushed stone flexible base
63-233-R	18	4	14	3.3	1.92	24	28	0	4	20	30	38	53	64	71	76	81	86	89	90	96	98	2.70	Crushed stone flexible base
58-232-R	29	11	16	7.2	1.84	15	32	0	21	45	56	64	73	80	83	85	86	87	88	89	94	99	2.70	Crushed stone flexible base
65-67-R	30	9	20	5.0	1.68	29	37	0	5	18	29	42	54	63	68	71	73	75	80	85	94	96	2.67	Crushed stone flexible base
65-100-R	24	9	15	4.7	1.89	14	19	0	10	19	31	46	62	73	82	86	88	90	92	93	97	99	2.76	Crushed stone flexible base
63-282-R	21	7	14	4.4	1.92	17	33	0	9	28	40	57	70	76	80	83	85	87	89	90	93	98	2.70	Crushed stone flexible base
67-194-R	19	4	16	2.2	1.85	22	32	0	9	20	36	52	63	68	72	78	83	87	90	90	95	99	2.67	Gravel flexible base
69-208-R	28	9	21	3.7	1.70	90	—							0	3	10								Caliche subgrade
69-209-R	26	5	22	2.2	1.65	75	—							0	11	25								Caliche subgrade
*66-154-R	21	4	19	2.2	1.83	27	37	0	10	18	26	37	51	63	70	73	75	78	80	82	95	98	2.74	Flexible base
*68-117-R	35	8	28	3.2	1.55	26	45		0	19	26	38	53	61	69	74	77	81	88	90	96	99	2.66	Caliche flexible base
*69-42-R	30	12	19	5.5	1.78	25	45	0	1	14	25	38	53	65	71	75	77	78	80	81	90	96	2.73	Flexible base

*Degradation tested materials.

then

1. Percent density $= \dfrac{\dfrac{DW}{V_{dh}}}{\dfrac{DW}{V_{DA}}} 100 = \dfrac{DW}{V_{dh}} \times \dfrac{V_{DA}}{DW} 100 = \dfrac{V_{DA}}{V_{dh}} 100$

2. $V_{DA} : V_G : : \dfrac{1}{D_A} : \dfrac{1}{D_G}$

3. $V_{DA} = \dfrac{D_G}{D_A} V_G$

4. Substituting step 3 in step 1, percent density $= \dfrac{\dfrac{D_G}{D_A} V_G}{V_dh} 100 = \dfrac{KV_G}{V_{dh}} 100$

Plotting of optimum wet densities of D_G versus D_G/D_A shows that for most all subbase and base materials this ratio, K, averages 1.05. Therefore,

5. Percent density for subgrade soils may be computed the same as in step 4, except the numerator factor K will vary between 1.05 and 1.23. This value should be determined from our standard correlation curve relating optimum wet density, D_{G_W}, to the ratio of D_G to D_A.

The following is a brief outline of the procedure for determination of the percent of compaction ratio density.

1. Dig a specified size test hole to determine volume of hole, V_{dh}.
2. By use of the gyratory press, run moisture-density curve for flexible base materials on the same material excavated by starting on dry side. (See separate gyratory procedure in the Appendix.) Break up soil and add moisture after each gyration until a 3- or 4-point curve is obtained. Mold all subgrade soils (except clean sands) in gyratory press at moisture contents below plastic limit. Determine minimum optimum volume V_G.
3. From relations given in Figure 4, determine K value (usually between 1.05 and 1.25). If desired, 1.05 may be used for all granular base and subbase materials.
4. Substitute values of V_{dh} (from step 1), V_G (step 2), and K factor (step 3) in the formula

$$\text{percent of compaction ratio density} = \frac{KV_G}{V_{dh}} 100$$

Many people have requested information concerning both the definition of compaction ratio density and a comparison of gyratory densities with those that might be obtained from AASHO test methods T99 and T180. The compaction ratio density values are different from those obtained by use of any of the three test methods, and yet they are calculated by use of some of these test results. Actually compaction ratio density is not solely a test method but rather is a calculated value utilizing several test methods. In its simplest terms, compaction ratio as used by us is the degree to which it is desirable to compact soil materials. The degree or change in density is based largely on the compactibility and volume change characteristics of the material.

Compaction ratio (C. R.) is expressed by the following formula:

$$\text{C.R.} = \frac{D_A - D_L}{D_D - D_L} \times 100$$

where

D_A = the dry weight per cubic foot of material to be obtained in the roadway;
D_D = the optimum dry weight per cubic foot obtained by running a moisture-density curve using a compactive effort of 30 ft-lb per cu in. (note that D_D is approximately equal to the value obtained by the modified AASHO method); and

$$D_L = \frac{\text{shrinkage ratio} \times 62.5}{1 + \dfrac{\text{LL - shrinkage limit}}{100}\ (\text{shrinkage ratio})}$$

D_L is obtained for base materials by use of standard dry rodded unit weight test and for soils is expressed as unit weight in pounds per cubic foot (pcf).

Figure 5 shows the approximate relationship between gyratory densities and those densities that might be expected from compaction ratio, AASHO T99, and AASHO T180. Generally speaking it may be noted that the compaction ratio D_A for swelling soils (they usually have low densities) is about the same as those to be expected from AASHO T99, but as unit weights increase compaction ratio densities approach those to be expected from AASHO T180. The equivalent line in Figure 5 indicates that the compaction produced by use of the proposed gyratory method is several pounds per cubic foot in excess of even that of AASHO T180.

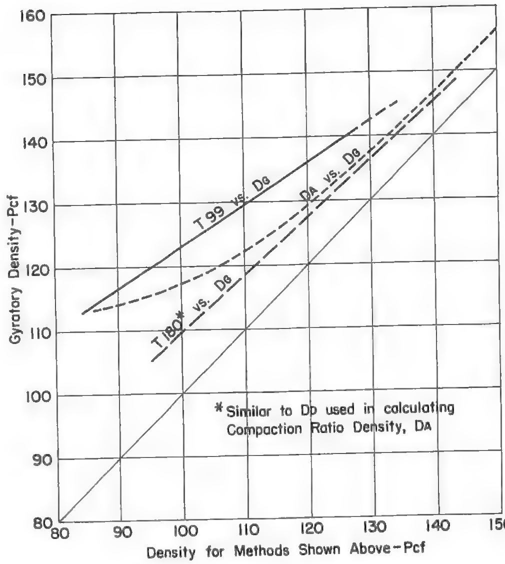

Figure 5. Relationship of gyratory density to other types of densities.

The basis for the gyratory effort used is largely empirical. It was desired to gyrate the material to its maximum density at 60 psi vertical pressure and later surcharge it with a static pressure for consolidation and/or leveling. Also, it was felt that some gyration at lower pressures might be helpful to begin to reorient the particles for movement. Thus, 20 and 40 psi pressures were used for a duration of 5 minutes each. Then final gyration at 60 psi vertical pressure was used until the hydraulic pressure gage showed no loss of pressure (no specimen densification) for 5 consecutive revolutions. Subsequent investigation showed that 2 minutes at the two lower pressures was sufficient. Because all soils do not compact at the same rate, it was felt inadvisable to gyrate for a particular length of time or a specified number of revolutions, but rather to gyrate to the specified end-point of a constant gage pressure for 5 revolutions of the machine platen. When the densities produced by these procedures were found to be some 6 to 10 pcf more than those of T180, few, if any, changes were made.

The use of repetitive gyration efforts led us to study degradation produced on three crushed materials having varying degrees of hardness. Results of gradation tests for 1 to 4 cycles are shown in Figures 6, 7, and 8. The degradation produced is so extensive that gradation data taken after gyration would very likely be misleading. The same statement does not apply to use of the gyration process for density control work. It may be observed in Figures 9, 10, and 11 that moisture-density relationships obtained for each material were little affected by degradation. This is believed to be due to the use of a gyratory action, coupled with a great compactive effort that produces degradation throughout a wide range of sizes, so that the overall density is not greatly affected. The extent of degradation of each stage appears to depend on rearrangement of particles to produce similar densities. Apparently, when the maximum density of a given moisture content is produced by the gyration method, degradation ceases to increase appreciably. When the material degrades, the voids between the aggregates contain more fines, which can be compacted more densely than before degradation. This is because the coarse aggregates tend not to hold the fines as far apart as they did before degradation. The gain in the density of the fines appears to be offset by a corresponding loss in the density of the coarser portions of the sample.

Tests on the material from Figure 6, but on a relative gradation of material all passing the $^3/_8$-in. sieve (which would represent tremendous degradation), were within 2 pcf of the peak of the curve for the flexible base gradation.

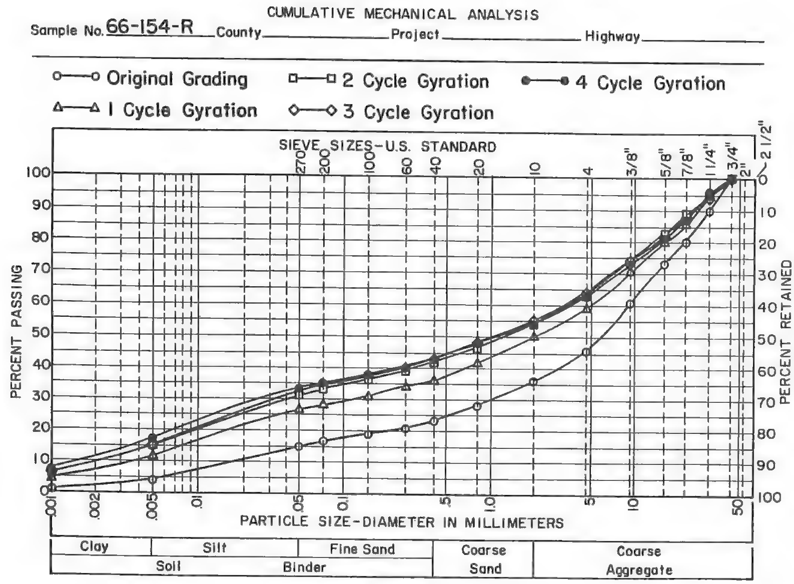

Figure 6. Degradation of particle sizes due to gyratory molding cycles, sample 66-154-R.

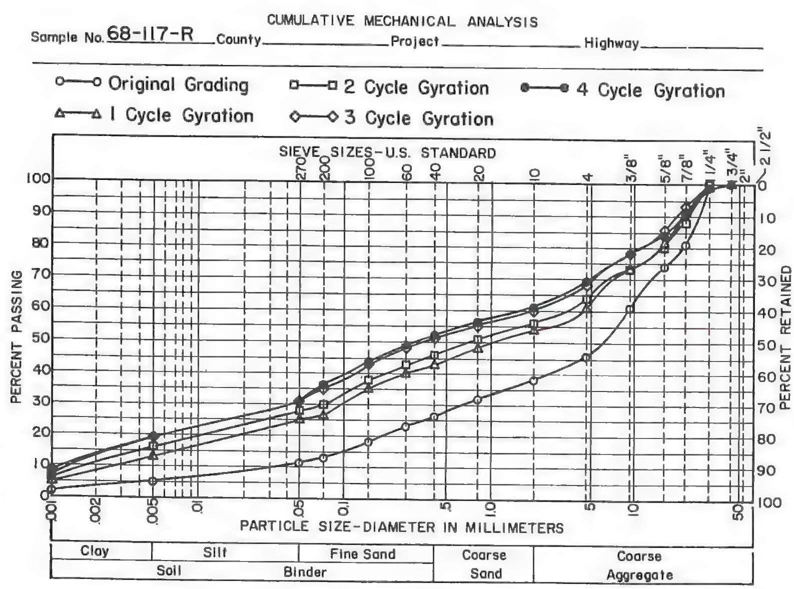

Figure 7. Degradation of particle sizes due to gyratory molding cycles, sample 68-117-R.

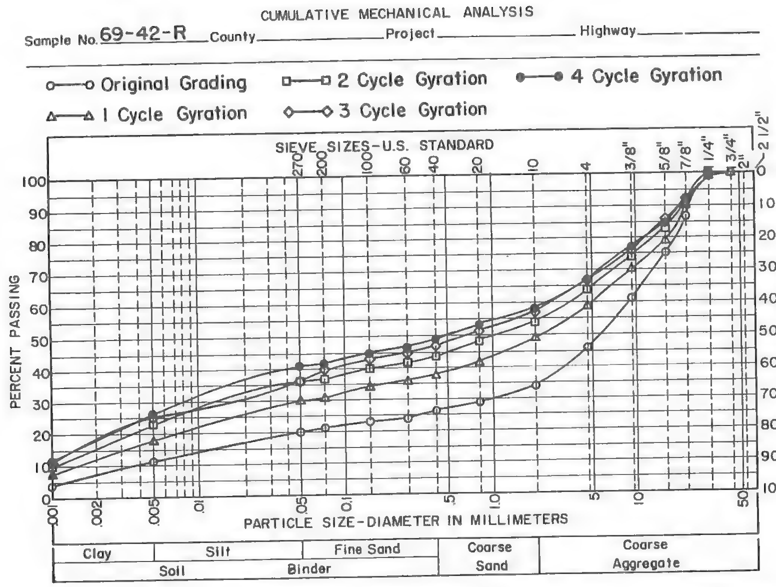

Figure 8. Degradation of particle sizes due to gyratory molding cycles, sample 69-42-R.

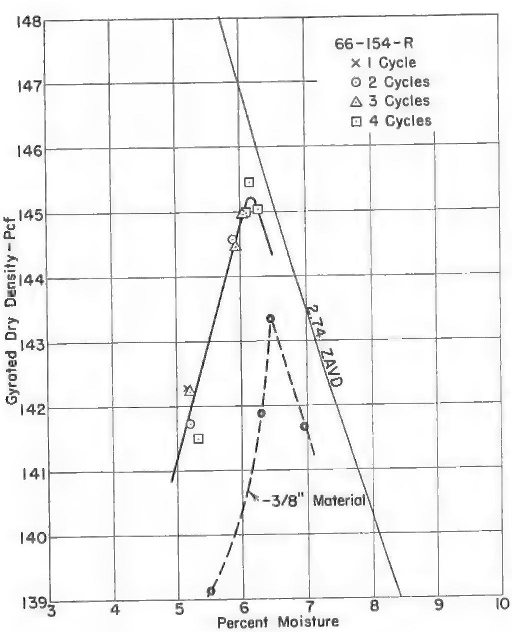

Figure 9. Effect of recycling of gyratory molding on moisture density, sample 66-154-R.

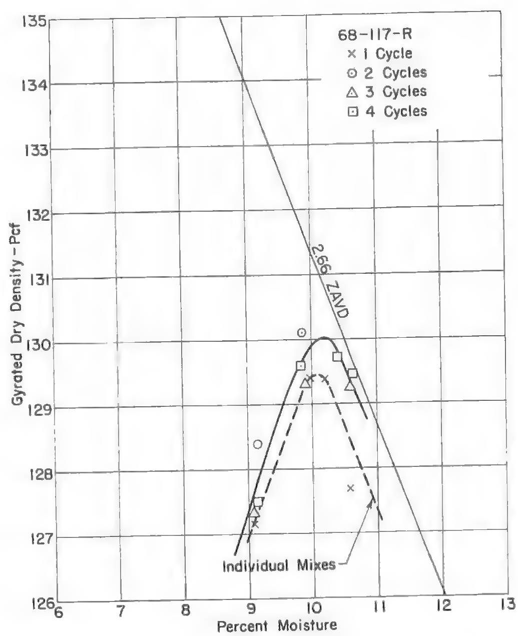

Figure 10. Effect of recycling of gyratory molding on moisture density, sample 68-117-R.

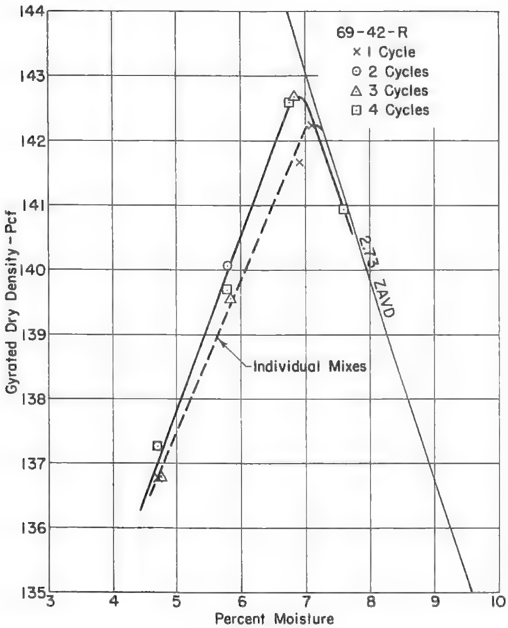

Figure 11. Effect of recycling of gyratory molding on moisture density, sample 69-42-R.

The 1-cycle experiments were extended by molding additional specimens for dashed lines shown in Figures 10 and 11. It may be noted that optimum moisture and density are fairly close to that obtained by a curve through all variable cycle points.

CONCLUSIONS

The investigation and subsequent experience appear to justify the following conclusions:

1. A method has been developed for density control of soil materials that has the following advantages over some other methods now in use: (a) The method expresses density control as a percentage of a density value that has been used successfully to control compaction of most highway, subgrade, subbase, and base construction in Texas. (b) Percent density is determined from material excavated from or near density test holes, thereby eliminating the old question as to whether or not the proper master moisture-density curve can be selected. Use of the proposed method will enhance the effectiveness in density control of erratic materials. (c) The method does not require the performance of preliminary moisture-density curves. (d) For materials that can be broken up with an ice pick (most granular materials), the method does not require drying out of the density test hole sample to determine percent compaction. (e) The method can usually be performed in approximately 1 to 2 hours, which is considerably less than the time required by other methods where data for preliminary moisture-density curves need to be redetermined. Other methods referred to include our presently used compaction ratio, standard AASHO procedures, and modified AASHO procedures, all of which will usually require 1 to 2 days to obtain new moisture-density curves. The proposed method is more specific and possibly more desirable than the other methods previously mentioned in that the same sample is measured for density both on the road and in the laboratory.
2. The method will be implemented in Texas because, since the initial writing of this report, 21 of the large gyratory compactors have been manufactured and are in service and 10 more will be available soon. The Manual of Testing Procedures has been enlarged to include the techniques discussed in this paper as an official testing procedure.
3. At the present time these techniques are limited to the extent that they are not applicable to cohesionless sands, soil-cement, soil-lime, or soil-asphalt.

ACKNOWLEDGMENTS

Acknowledgment is made to members of the Texas Highway Department, Materials and Tests Division, under the leadership of A. W. Eatman, Materials and Tests Engineer. The performance of many tests and their presentation by members of the THD Soils Section of the Materials and Tests Division is gratefully acknowledged.

REFERENCES

1. McDowell, Chester. Selection of Densities for Subgrades and Flexible Base Materials. HRB Bull. 93, 1954, pp. 1-15.

2. McDowell, Chester. Principles of Controlled Compaction. WASHO Proceedings, 1957.
3. McDowell, Chester. Density Control: Its Benefits and Complexities. THD TP 4-66-E, Sept. 1966; Highway Research Record 177, 1967, pp. 197-218.
4. McDowell, Chester. Interrelationship of Load, Volume Change, and Layer Thicknesses of Soils to the Behavior of Engineering Structures. HRB Proc., Vol. 35, 1956, pp. 754-772.
5. McDowell, Chester. Remedial Procedures Used in the Reduction of Detrimental Effects of Swelling Soils. WASHO, June 1965; Concluding Proceedings, International Research and Engineering Conference on Expansive Clay Soils, Texas A&M Press, 1965.

APPENDIX

MOLDING FLEXIBLE BASE, SUBBASE, AND SUBGRADE SOIL MATERIALS USING THE LARGE GYRATORY COMPACTOR AND DETERMINATION OF PERCENT D_A

This test method is intended to be used for determining the percent D_A, i. e., the actual desirable density to be achieved in the roadway structure, from density samples taken from the roadway. These samples may be base, subbase, or subgrade soils up to a top size material that passes the 1¾-in. sieve. It is intended that density hole samples be taken in the field, transported without loss of moisture, and gyrated in the large gyratory compactor where a minimum volume will be obtained. The relationship of the minimum volume, V_G, and the volume of that same material when in the density hole has been correlated to give the percent D_A.

Because this test is based on volume of the same soil in two places and conditions, it is necessary that the dry weight of the soil remain constant (no loss). If a small amount of soil is lost during gyration, a correction should be applied. The test method, in its simplest form, consists of comparing minimum gyratory volumes with the volume of that same soil taken from roadway density test holes and is not applicable to cohesionless sands, soil-cement, soil-lime, or soil-asphalt.

Definitions of Symbols Used in Procedure

D_A = actual density, in dry weight per weight per cubic foot of material, to be obtained in the roadway.

D_G = highest density, in dry weight per cubic foot of material, obtainable by gyrating a raw soil in the 7- by 3-in. gyratory size using water as a lubricant and gyratory procedure similar to that used here.

V_{dh} = volume of the density hole determination made in the field for the soil in question being tested.

V_G = minimum volume of a soil corresponding to D_G.

Optimum gyratory moisture = the moisture content of the soil at D_G.

V_{DA} = volume of material when at D_A density.

D_{GW} = optimum wet density of the gyrated material at D_G and V_G.

DW = dry weight of soil.

Apparatus

1. Gyratory compactor—A motorized compaction device (Fig. 12) capable of gyrating 6-in. I. D. by 12-in. high forming molds. The compactor shall be capable of gyrating at an angle of approximately 5 deg and shall have a hydraulic ram capable of maintaining a load of 500 psi on the gyrated specimen.

2. Compaction mold—A 7-in. ±0.030 in. I. D. by 7 7/16-in. ±1/16 in. high forming mold with gyratory flange collar and mold base plate (Fig. 2). It is preferable, but not necessary, that the mold be chromium-lined.

Figure 12. Motorized gyratory press.

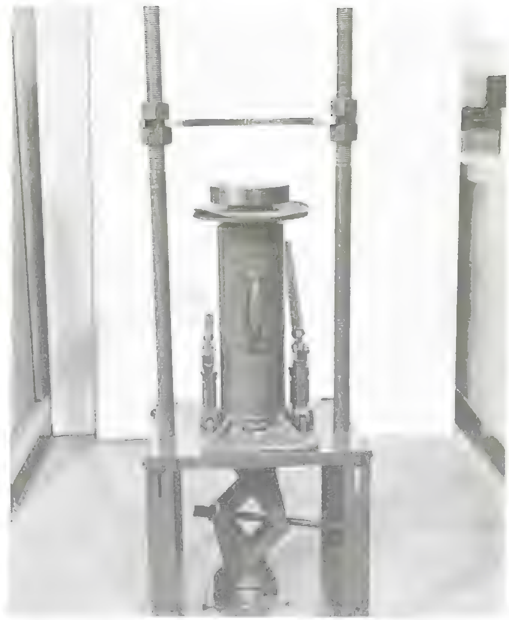

Figure 13. Extrusion press.

3. Measuring device—A micrometer dial assembly for determining height of specimens, with set of standardized spacer blocks.
4. Scale—Rated 30-lb capacity and sensitive to 0.01 lb.
5. Press—Ejects specimens from mold (Fig. 13).
6. Electric hot plate.
7. Dolly—Caster-mounted, made to same height as compactor platen and extrusion press platen (Fig. 14).
8. Metal pans—Approximately 21 by 15 by 4 in., used for drying and mixing materials.
9. Metal disks—7-in. diameter by approximately 0.040-in. thick (18 gage sheet metal).
10. Filter paper—7-in. diameter.
11. A supply of small tools—trowels, ice pick, plastic mallet, etc.
12. A supply of plastic bags, approximately $9\frac{1}{2}$ by 18 in., for retaining test samples in their natural moisture condition.

Figure 14. Caster-mounted dolly.

Test Record Form

Record test data on work sheet (Fig. 15).

Calibration of Equipment

Specimens are molded to a height of approximately 3 in. in the 7-in. diameter mold. Because the compacted specimen does not completely fill the mold, it is necessary to determine the volume per linear inch of height of the mold. Determine this factor as follows:

1. Measure the diameter of the mold, by means of a micrometer caliper and micrometer dial, at the ends and several intermediate points to obtain an accurate average value for the diameter.
2. Use the average diameter to calculate a mean cross-sectional area of the mold.
3. Calculate the volume in cubic feet for 1 in. of height of the mold as follows:

$$\text{Volume of mold, cubic feet per inch} = \frac{\text{area in square inches} \times 1 \text{ inch}}{1{,}728}$$

Using the micrometer dial assembly and an appropriate 3-in. spacer block, place the mold base plate in position on the gyratory platen. Then bring the top gyratory head down on the spacer and, using the hydraulic pump mechanism, bring the total load to 19,240 lb (500 psi on a 7-in. diameter specimen). Determine the dial setting for a 3-in. high specimen. Set the dial to read zero. Specimens higher than 3 in. will read greater than the "zero" reading; smaller specimens will read less.

Securing Samples for Testing

This test differs somewhat from other soil tests insofar as a key or master sample is taken, and then the adjacent material is excavated for a work sample. Proceed as follows:

1. Upon selecting the point to be sampled, dig a density test hole about 0.07 cubic foot in size and accurately determine its volume. Seal the contents dug from the hole in a plastic bag to prevent drying out. Attach proper identification to this bag and mark it "key" or master sample.
2. Then secure at least two more plastic bags full of soil by enlarging and/or deepening the test hole. Care should be used when deepening a hole to avoid material that is an obvious change of layers. Seal these bags as before and identify as "work" samples.

Lab No.________

____% Hygro Allowed

RAW SOIL GYRATORY WORK SHEET

Date Molded															
Sample No.															
Total % Water															
Pounds Material (Lab)															
Pounds Water Desired															
Pounds Hygroscopic Water															
Pounds Water Added															
Tare Wt. of Jar															
Wt. Jar and Water															
Mold No.															
Time @ 20 psi															
Time @ 40 psi															
Time @ 60 psi															
Time @ 500 psi															
Ht. Specimen and Plates															
Thickness of Plates															
Ht. Specimen			REDUCED SIZE - TOO SMALL FOR RECORDING DATA												
Pushed Out Ht.															
Wet Wt. Specimen															
Vol. per Linear Inch															
Vol. of Specimen															
Wet Density Specimen															
Dry Wt. Pan & Specimen															
Tare Wt. Pan															
Dry Wt. Material															
Weight Water															
Percent Water on Total															
Dry Density															
Guestimated Dry Density															
Wet Wt. Key Sample (Field)															
Vol. Density Test Hole (V_{dh})															
Max. Gyrated Density (D_G)															
Vol. Specimen @ D_G (V_G)															
Wet Density @ D_G (D_{GW})															
K Constant															
Percent D_A Density															

Figure 15. Work sheet.

Gyratory Molding of Samples

The use of plastic bags of a uniform thickness and size will be very helpful because they will have a uniform tare weight. Proceed with preparation and molding as follows:

1. Determine the wet weight of the key sample. Record this weight on the data sheet (Fig. 15).
2. From the work sample, take an ample and representative sample for a moisture determination. Place this sample in a shallow pan and dry on an electric hot plate. Determine its moisture content and use this percentage as the moisture content for the key sample.
3. For fine-grained soils and soils with low percentages of aggregates, take a specimen for molding equal in weight to the wet weight of the key sample, from the work sample bags.
4. For flexible base or subbase material that can be broken up with an ice pick, the key sample may be used, if desired, for molding, rewetting, and remolding if the gyratory curve begins on the dry side and subsequent remoldings are successively wetter.
5. Either by drying or by moistening, as may be required, prepared the soils using steps 3 and 4 for gyrating to produce V_G and D_{GW}. Note: Many fine-grained soils, which will be controlled by compaction ratio methods, have their moisture content for D_G between 11 and 17 percent moisture. All fine-grained soils, with little or no aggregates, should have D_G values with moisture contents below the PI. If a good estimate of the PI can be made, then the moisture content of the first "work" specimen can be reasonably close to the optimum. Base and subbase materials at D_G condition look somewhat drier than moisture contents for triaxial molding before loading in the 7- by 3-in. gyratory mold, and most exhibit a trace of a moisture bead between the inside of the mold and the top gyrating head under 500-psi load. It is intended for fine-grain soils to use the "work" samples to bracket the optimum gyratory conditions and then, wet or dry, the "key" sample, as may be required, for the optimum or maximum density, D_G. For base or subbase, or other granular aggregate materials that can be broken down and rewetted, begin on the dry side and add increments of water for successive remoldings. It should be noted that any soil lost during gyration should be accounted for, and a correction applied, because this test assumes a constant dry weight (density hole and gyratory mold).
6. Prior to loading the specimen in the mold for gyrating, place it in a mixing pan and make uniform by stirring, blending, and in the case of fine-grain soils passing it through a ¼-in. hardware cloth where possible. Load the specimen in the 7- by 3-in. mold as follows: (a) Place the mold and base plate (bottom gyrating head) in place, and move the dolly to the specimen to be loaded. (b) Place a 7-in. disk of 18-gage metal in the mold and on top of the bottom gyrating head. Place a filter paper on top of the metal disk. (c) Place about a ½-in. depth of loose material, from the finer sizes, in the bottom of the mold. (d) Then place any aggregates and intermediate sizes, spading around the sides of the mold with a putty knife or spatula. (e) Level out the aggregates and place the remaining fines, leveling them out. Then put a filter paper in the mold on top of the leveled layer, and then a 7-in. methal disk. (Note: Fluffy, lightweight fine-grained soils, which tend to overfill the mold when loaded, should be pushed down lightly with a flat finishing tool enough so that the fluffy material can be placed in the mold.) (f) Move the dolly, with mold and contents, to the gyratory machine.
7. Slide the mold, with base plate, onto the platen of the compactor. The platen must have a generous coat of good lubricant or the platen and base plate can be damaged. Center the mold, lower the compactor head on the material, and turn the lift cam down to give the mold its ¾-in. lift angle.
8. Using the machine controls, place a load of 20 psi with the loading ram on the specimen and turn on the machine. Gyrate the specimen for 2 minutes at 47 psi gage loading (47 psi gage is 20 psi on the specimen).
9. At the end of 2 minutes, increase the load to 94 psi gage and continue gyrating 2 minutes at 40 psi on the specimen.

10. Then increase the load on the ram to 141 psi gage and continue gyration until the gage needle will stand steady on 60 psi on the specimen for 5 revolutions of the platen. This means that there has been no appreciable shortening or densification in the five revolutions. Turn the compactor off.

11. Release the pressure from the top of the specimen slightly and, using the handle provided, return the cam lift to its original position and reduce the angle of lift to zero.

12. Place 47 psi gage pressure on the specimen and turn the machine on for a few revolutions. This tends to square-up the specimen. Turn the machine off.

13. Wipe off any oil on the platen and place 1,177-psi gage pressure on the specimen. This is 500 psi on the cross section of the specimen.

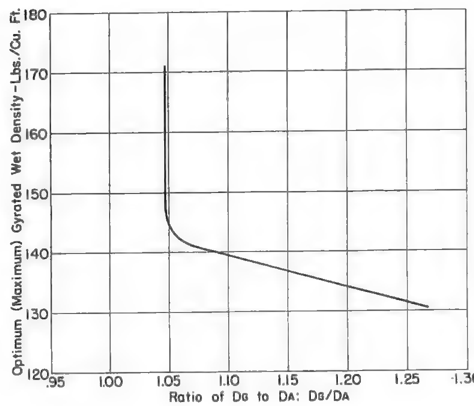

Figure 16. Optimum gyrated wet density versus D_G/D_A (K factor).

14. Place the pre-set measuring stand in position to measure the height of specimen (Fig. 1). Hold the load on the specimen until the rate of consolidation is 0.005 in. or less in 5 minutes.

15. Observe the dial reading and record the net height of specimen only, making allowance for the thickness of the metal disks.

16. Remove the measuring device and then the load on the specimen. Raise the ram out of the mold and remove the mold from the machine platen to the dolly.

17. Slide the mold with base plate on the platen of the ejection press (Fig. 13) and eject the specimen up and out of the mold.

18. Remove the top and bottom metal disks and weigh the specimen to the nearest estimated 0.001 lb. Clean any material adhering to the disks that was included in the measured height and weigh this with the specimen. If any appreciable material was lost during gyration it should be recovered and a correction for volume made on the assumption that it would have been densified the same as was the specimen itself.

19. Calculate the volume and density of the specimen.

20. In the case of flexible base or granular aggregate materials, use an ice pick or other convenient tool to break up the specimen completely, add a desired increment of water, and mix, load, and gyrate as before until D_G is determined. Usually three points will suffice for these data.

21. In case the material is a fine-grained or other subgrade soil containing few aggregates, use the second "work" specimen, molding it just wetter (1 or 2 percent) than where optimum appears to be. Mix, load, and mold as before. When the optimum condition has been bracketed using the "work" sample, take the key sample and mix it at optimum conditions and load and mold it to determine D_G. Note: The zero air voids density line, calculated from the specific gravity of the material, is useful, as in all moisture-density curves. At the optimum conditions (the peak of the curve), most materials show an average of 2 percent air voids at its optimum moisture.

Calculations

Calculate V_G, D_{GW}, and percent D_A as follows:

V_G = height of specimen under load × calibration factor of mold

$$D_{GW}, \text{ at optimum conditions} = \frac{\text{wet weight of specimen}}{V_G}$$

$$\text{Percent } D_A = \frac{K V_G}{V_{dh}} 100$$

where K, constant, is taken from Figure 16.

THE USE OF AN IMPACT ROLLER IN COMPACTING A COLLAPSING SAND SUBGRADE FOR A FREEWAY

A. A. B. Williams, National Building Research Institute, Pretoria, South Africa; and
G. P. Marais, Transvaal Roads Department, Pretoria, South Africa

The problem of differential settlement on roads in southern Africa has occurred on certain silty sands with high void ratios, due to the "collapse" of the soil structure. In comparative field trials of possible solutions to this problem, a flat-sided impact roller was found to cause consolidation to greater depth than vibrating or pneumatic rollers. This impact roller was later used for the compaction of a silty sand subgrade under a new freeway near Witbank. Three deep profiles were then studied: under the freeway where the impact roller had been used, under an old existing asphalt road, and in the undisturbed natural condition. The variations with depth of density, moisture content, and penetration resistance were determined. "Double oedometer" tests were carried out in the laboratory on undisturbed samples, making possible the prediction of collapse settlement on inundation under various loads. It was found that the state of compaction achieved by the impact roller was sufficient to prevent any later collapse settlement with normal traffic loads under conditions of inundation. Further, it was concluded that special measures should be taken to compact such subgrades to a depth of 1.4 meters (4 ft 6 in.), in order to prevent collapse settlement under heavy traffic.

•IT has been known for some time that structural damage to buildings and roads in southern Africa occurs on certain silty sands possessing high void ratios (commonly known as "collapsing sands") because of the "collapse" of the soil structure. There have been several publications since the first by Jennings and Knight in 1957 (1), but the nature of the problem is not fully understood. The full scope of the problem in the Transvaal province alone is not known, but difficulties could well be experienced in some parts of those areas of sandy soils containing iron oxides, which are referred to on the soil map of southern Africa as "lateritic soils" or "ferruginous lateritic soils." There are many other parts of the Republic where loose windblown sands exist to considerable depths, particularly in the northern Cape, Orange Free State, coastal Natal, most of South West Africa, and Botswana. The severity of this effect on a major road was referred to by Knight and Dehlen in 1963 (2) when it was shown that a settlement at the surface of about 150 mm (6 in.) had occurred and that there was evidence of some densification under traffic to a depth of about 1.3 meters (4 ft 6 in.). It had been noted by Williams (3) that any soil with a dry density of less than about 1,600 kg/m^3 (100 lb/ft^3) could give rise to difficulties in settlement and that if a density of 1,450 kg/m^3 (90 lb/ft^3) existed, the possibility of collapse was very likely.

When the layer of windblown sand is less than a meter thick, there is little problem in carrying foundations for a building down to a hard stratum, or in compacting this layer for a road foundation. When the deposit is thicker than this, some special treatment is required in both cases. In other parts of the world deep loessial deposits give rise to similar problems, although the material is much finer than these uniform sands.

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THE ROAD PROBLEM

The construction of a new freeway traversing areas of collapsing sand in the Witbank area 100 km (60 miles) east of Pretoria was begun in 1968. The section of freeway concerned is 16 km (10 miles) long and accommodates 3,000 to 4,000 vehicles per day. There are two traffic lanes in each direction, and the shoulders are surfaced. The value of the 3-year contract awarded for construction was R5.5 million ($7.7 million). The pavement has been designed by the interim AASHO method to a "structural number" of 4.5 and is composed of the following layers:

1. 50 mm (2 in.) of asphaltic concrete;
2. 150 mm (6 in.) of cement-treated crushed stone base, 5.0 MN/m^2 (750 psi);
3. 150 mm (6 in.) of cement-stabilized lateritic gravel subbase, 1.7 MN/m^2 (250 psi);
4. 150 mm (6 in.) of imported subgrade compacted to 95 percent modified AASHO density; and
5. 450 mm (18 in.) of in situ or imported sand subgrade compacted to 95 percent modified AASHO density.

At the time the contract was drawn up, it was recognized that there might be a problem of possible settlement; however, the only proven method of compacting the in situ sand subgrade was by removing 600 mm (24 in.), compacting the exposed surface with a heavy pneumatic or vibrating roller, replacing the excavated material, and then compacting this to 95 percent modified AASHO density. This procedure proved almost unworkable in practice. The scrapers repeatedly got bogged down, often due to the high water table in some areas, and even the lowering of the water table by means of drains had relatively little effect.

There was thus not only the immediate practical problem of how to achieve compaction economically but also the doubt about what depths and limits on density should be specified in this deep loose sand subgrade.

FIELD INVESTIGATION

The foregoing problems had received little attention in South Africa until during the course of the Witbank road contract some evaluation trials were carried out on a new type of pentagon-shaped roller that appeared to be a very feasible solution to the problem of compaction in depth. The action of the roller (Fig. 1) delivers a series of impacts as its 11,000-kg (24,000-lb) mass drops about 200 mm (8 in.) onto a rectangular area 1.2 by 0.6 meter (4 x 2 ft). While operating at about 8 km/h (5 mph), this applies impact loads every 0.75 meter (30 in.) at a frequency of about 3 cycles per second. The results of this work reported by Clegg in 1969 (4) showed conclusively that the impact roller achieved an appreciably better effect than does a heavy pneumatic or vibrating roller and had a high work output. In the course of this work, it was found that the use of a small cone penetrometer proved very useful for following the progressive change in condition of a deep sand subgrade as rolling proceeded. Some further studies were reported by Williams in 1970 (5); these confirmed that the effect of the impact roller was much greater down to a depth of about 1.5 meters (5 ft) than any of the other rollers and gave information on the pressures generated through the soil profile.

Figure 1. The impact roller about to deliver a blow.

It was not known, however, if the state of compaction achieved under the impact roller was sufficient to prevent collapse settlement later with normal traffic loads as a result of inundation. The problem could be framed differently: to enquire whether the over-consolidation achieved by the impact roller at the field moisture content would prevent further compression of the sand on inundation under the traffic loading. A further field and laboratory study was thus planned to seek information on these points.

Three sites were selected for the study. They were located some distance east of where the roller trials reported by Clegg had been carried out. Site A was considered to be undisturbed by any heavy equipment because it lay in an old orchard. Site B was under the old road, which had carried normal traffic for more than 25 years. Site C was on the new freeway, complete except for base and surfacing. The selected subgrade layer had been subjected to twelve passes of the impact roller. It had been observed by the resident engineer that during construction considerable additional settlement of the surface had occurred when the impact roller was used after normal rolling (6).

It was intended that a comparison be made between conditions existing before any treatment was given to the soil and those finally obtained under the impact roller. It was also hoped that the conditions under the old existing road would reveal whether densities such as those produced by many years of trafficking were attained by the use of the impact roller on the selected subgrade.

The dynamic cone penetrometer described by van Vuuren in 1969 (7) had been found to give a useful indication of the state of compaction of the collapsing sand after various passes of the impact roller. This device simply consists of a 10-kg mass falling through 460 mm onto a 20-mm diameter point. The penetration resistance of the point with depth was therefore determined at each site. At each site, two test pits 2.1 by 2.75 meters (7 by 9 ft) were excavated carefully by hand to a depth of 1.8 meters (6 ft). The soil profiles of the six holes are shown in Figure 2. Before excavation was started, the dynamic cone penetration resistance was measured to a depth of 2 meters (6 ft 6 in.) at each of the four corners of the test pits.

At vertical intervals of 300 mm (1 ft) the in situ density was determined by both the sand replacement method and a Hidrodensimeter nuclear moisture-density gage. Samples for moisture content determination and laboratory testing were also taken. Subsequently, undisturbed block samples were cut from the walls of the test pits for oedometer testing. The depths at which the samples were taken are shown in Figure 2.

RESULTS OF FIELD WORK

The variations in natural moisture content and dry density with depth are shown in Figures 3 and 4. The variation with depth of dry density as a percentage of standard AASHO is shown in Figure 5. The in situ "estimated CBR" values as determined with the dynamic cone penetrometer are shown in Figure 6.

ROUTINE LABORATORY TESTS

On each sample taken at 300-mm (1-ft) intervals in each hole, the moisture content was determined by oven drying, a grading analysis was carried out, and the moisture-density relation was determined.

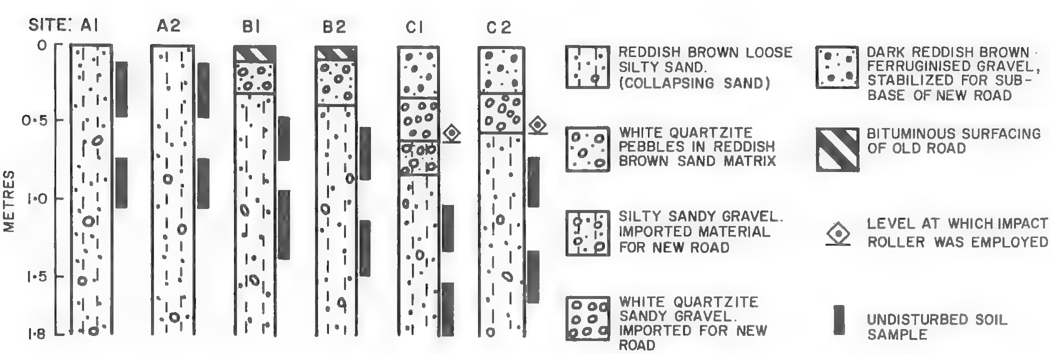

Figure 2. Soil profiles at field sites near Witbank.

The particle size distribution of all samples fell within a fairly narrow envelope, as follows:

U.S. Sieve Size	Percentage Passing
4	99 to 100
10	90 to 99
40	65 to 81
200	16 to 38

There was a tendency for the finest fraction to increase with depth, and the plasticity also increased from "non-plastic" in the upper horizon to a maximum plasticity index of 13 at a depth of 1.8 meters (6 ft).

Specific gravity was measured on the samples, and the tests gave an average specific gravity of approximately 2.66.

In order to throw new insight into the state of the material either in its natural condition or after compaction, it was thought that the use of Burmister's "relative density" concept (8) might prove rewarding. This term should not be confused with relative compaction, which is a ratio between the actual density and some maximum density achieved by standard methods such as modified AASHO. The relative density of Burmister is defined as

$$R.D. = \frac{e_L - e}{e_L - e_D}$$

where

e_L = void ratio of the soil in loosest state,
e_D = void ratio of the soil in densest state, and
e = void ratio of soil in question.

Tests were thus carried out by following the method specified by Akroyd (9). Method III.I. (ii) (2,000-ml cylinder method) was used for the minimum density, and method III.H. (i) (standard compaction mold method with vibration) was used for the maximum density.

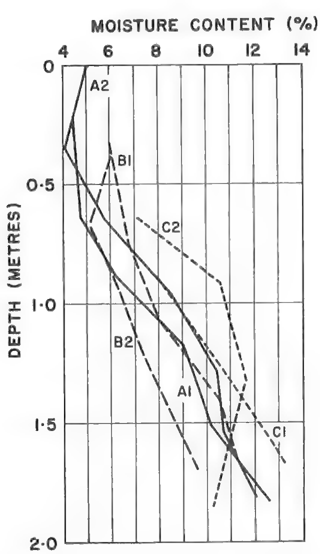

Figure 3. Natural moisture content profile.

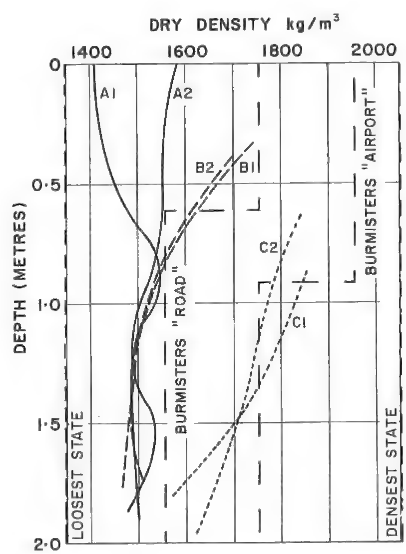

Figure 4. Dry density against depth.

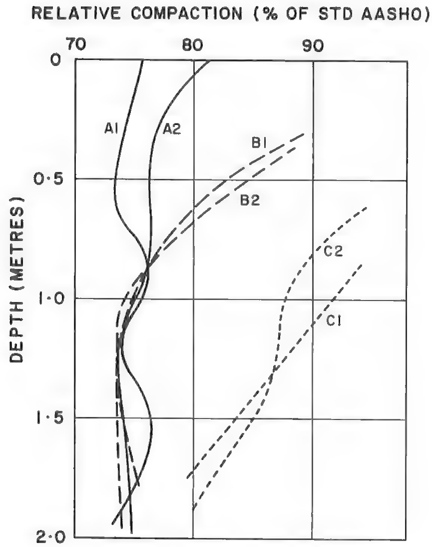

Figure 5. Relative compaction with depth.

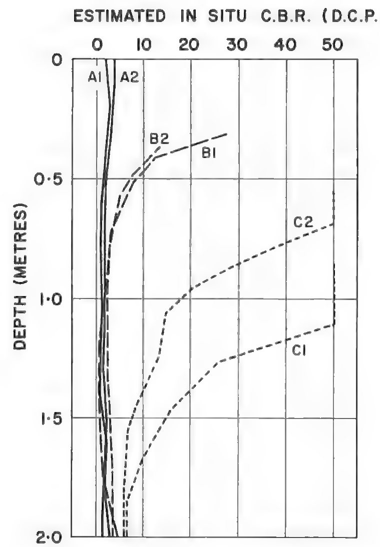

Figure 6. Estimated CBR values with depth.

The results may be summarized as follows: The minimum density averaged 1,365 kg/m^3 (85 lb/ft^3), a void ratio of 0.95, and the maximum density averaged 2,050 kg/m^3 (128 lb/ft^3), a void ratio of 0.30. There was very little difference in the values obtained between tests on all the six test pits. The values are shown in Figure 7, and this relative density diagram now allows one to read off the relative density for any value of soil density that may be encountered. This will be referred to later when comparisons are made between various field data.

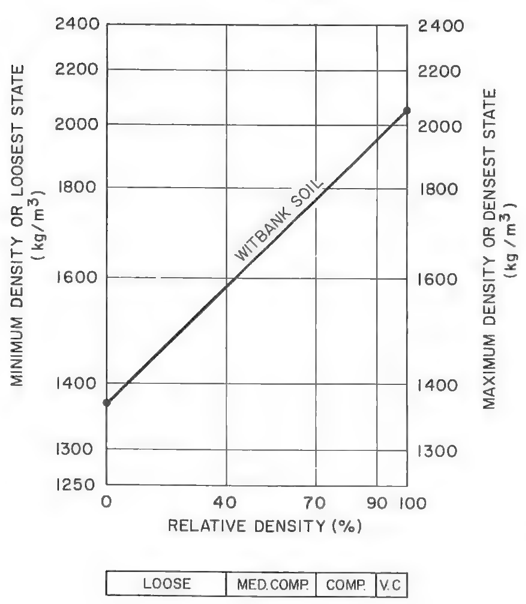

Figure 7. Relative density diagram.

DOUBLE OEDOMETER TESTS

Several oedometer tests were carried out on materials, in both wet and dry states, during the first investigation to study the consolidation characteristics, and the results can generally by summarized as follows:

1. Compression index, C_c = 0.20 to 0.30;
2. Preconsolidation load, P_c, dry = 70 to 90 KN/m^2 (10 to 13 psi); wet = 17.5 to 70 KN/m^2 (2.5 to 10 psi);
3. Initial void ratio, e_o = 0.85 to 0.95; and
4. Void ratio at 4,425 KN/m^2, e_f = (say) 0.30.

For the sites nearer Witbank further "double oedometer" tests were carried out, and typical results are shown in Figure 8. In these tests, undisturbed samples for testing in standard oedometers were trimmed into molds about 75 mm in diameter and 20 mm thick (3 in. in diameter, 0.75 in.

thick) from a larger block of soil carefully cut out in the field test pit. Initially the method of testing first suggested by Jennings and Knight (1) was used, in which two similar samples are loaded—one at the natural moisture content and one inundated with water. It was found that there was often little difference between the compression curves obtained, presumably because the natural moisture content was fairly high (about 10 percent in some cases) following rain. Because the potential "collapse settlement" was not clearly demonstrated, it was decided to allow samples to dry out in the laboratory atmosphere (about 50 percent relative humidity) and then carry out the two tests. This treatment showed up the potential collapse very clearly. It was noted that many of the in situ void ratios approached 0.95, which is equivalent to a density of 1,365 kg/m^3 (85 lb/ft^3), and that under the highest pressure possible of 4,425 kN/m^2 (640 psi), the void ratio approached 0.30, which is equivalent to a dry density of 2,050 kg/m^3 (128 lb/ft^3).

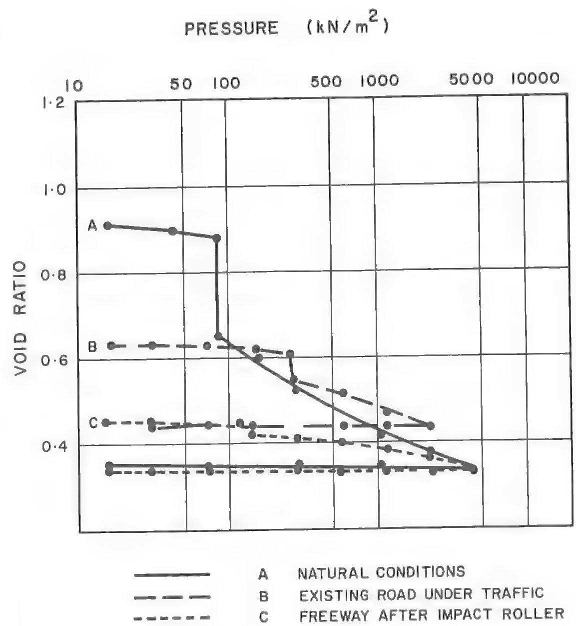

Figure 8. Typical double oedometer tests.

In general, three samples from each of two levels in the six pits were tested. One sample was loaded in the dry condition in increments up to 4,425 kN/m^2 (640 psi). The second was loaded to an applied pressure estimated from the pneumatic roller curve in Figure 9 for that depth and was then inundated before completing the loading cycle. The third sample was loaded to a pressure estimated from the impact roller curve in Figure 9 for that depth. This figure was obtained (5) from the results of a previous installation containing miniature earth pressure cells, and it will be noted that the impact roller generated a pressure of about 200 kN/m^2 at 1 meter depth (30 psi at 3 ft) compared with the heavy pneumatic roller giving 70 kN/m^2 at that depth (10 psi at 3 ft).

A very good idea could thus be obtained of the settlement behavior between the dry and saturated condition under different loadings that were near the range of loading that might be experienced under traffic.

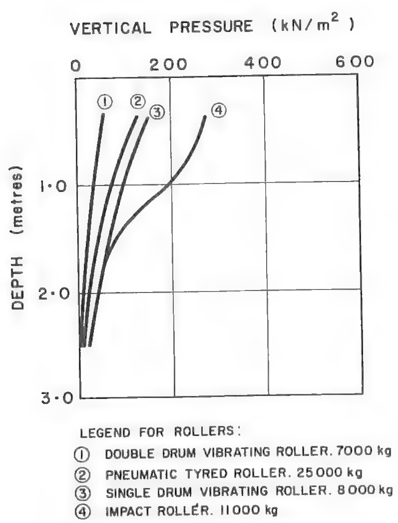

Figure 9. Vertical earth pressures under various rollers near Witbank.

INTERPRETATION OF TEST DATA

An idealization of the compression characteristics of the Witbank sands is shown in Figure 10; the characteristics are very similar to those described by Knight (10). He had also found that a family of compression curves could be obtained between the dry and completely wet conditions, depending on the mositure content. When a "critical moisture content" of about 15 percent was exceeded, however, the material no longer exhibited a sudden "collapse" but followed the virgin compression curve.

The potential collapse settlement of any element of soil beneath a road can be assessed (Fig. 10) once the void ratio of that element and the pressure to which it will be subjected by traffic loads are known. The void ratio at any depth in the soil profiles can be obtained from Figure 4; a rough approximation of the pressure distribution under a heavy wheel load may be obtained from curve 2 in Figure 9 for the pneumatic-tired roller. In this way the upper points shown in Figure 10 can be plotted to represent conditions under the old existing asphalt road (site B). The lower points in Figure 10 represent the void ratios that are achieved at similar depths after compaction with the impact roller (site C).

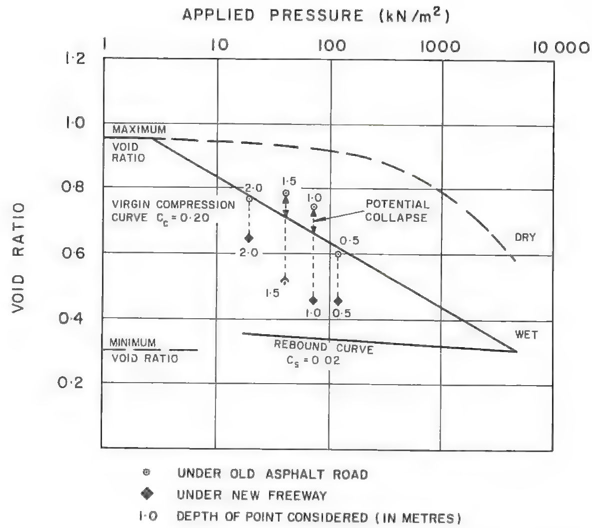

Figure 10. Compression diagram for typical collapsing sands at Witbank.

Thus, for example, at a depth of 1.0 meter (3 ft) beneath the old road the void ratio was 0.77. The material at this point might have been subjected to a pressure of about 70 kN/m^2 (10 psi), and reference to Figure 10 shows that this could have collapsed on inundation to a void ratio of 0.66 at the same pressure. At a similar depth of 1.0 meter under the new freeway, the impact roller would have preconsolidated the material to a void ratio of 0.46. Therefore no further settlement, or collapse, would occur under the pressures of about 70 kN/m^2 that might be generated by traffic. This is much the same for the points at other depths shown; therefore, the impact roller shows great promise for treatment of collapsing or, rather, highly compressible sand subgrades.

DISCUSSION OF RESULTS

It is thought that the use of the term "collapsing sand" may not be fully understood by some people in that, if the sand is wet, the actual phenomenon of sudden settlement (or collapse) will not occur. It should be emphasized, however, that considerable compression will still take place if the material is already wet. A better way to convey this meaning to those not fully aware of the subject, therefore, may be to emphasize that these loose windblown sands are very compressible. A coefficient of compressibility, C_c, of 0.2 to 0.3 is similar to that of some remolded clays, and it will thus be realized that appreciable settlement can occur purely through an increase in load under high moisture content conditions without complete inundation of these sands.

It has become clear from the relative density tests that the condition of the collapsing sand subgrades in the natural state approaches the loosest state of packing that can be achieved and that this condition persists for considerable depth. In regard to the depth to which compaction should extend, the early work of Burmister (8) suggested limits for roads and airport runways, which have been indicated in Figure 4. Further, it would appear that the recommendations made by Foster and Ahlvin (11) would also be applicable to collapsing sands and, in fact, are readily achieved by treatment with the impact roller.

CONCLUSIONS

From this and previous experience it would appear that specifications should cover the state of compaction to be achieved to depths of at least 1.4 meters (4 ft 6 in.) below final road level. With reference to the actual conditions on site, it is apparent that the impact roller has given a relative density that is higher than that achieved after many

years in the field when a road has been exposed both to heavy wheel loads and to conditions of wetting and drying through the normal climatic seasons. Further, the impact roller does offer economic advantages over other methods for achieving great depth of compaction in collapsing sands.

ACKNOWLEDGMENTS

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REFERENCES

1. Jennings, J. E., and Knight, K. The Additional Settlement of Foundations Due to a Collapse of Structure of Sandy Subsoils on Wetting. Proc. Fourth Internat. Conf. on Soil Mech. Found. Engng., 1957, Vol. 1, p. 316.
2. Knight, K., and Dehlen, G. L. The Failure of a Road Constructed on a Collapsing Soil. Proc. Third Reg. Conf. Africa on Soil Mech. Found. Engng., Salisbury 1963.
3. Williams, A. A. B. Discussion of paper. Trans. S.A. Inst. of Civ. Engrs., 1957, p. 121.
4. Clegg, B. Report on a Field Trial to Investigate the Performance of Different Types of Roller for Deep Compaction. NIRR unpublished report RC/3/1969, NIRR, CSIR, Pretoria.
5. Williams, A. A. B. The Use of Small Earth Pressure Cells in Some Road Experiments. NIRR unpublished report RS/3/1970, NIRR, CSIR, Pretoria.
6. Liell Cock, D. P. M. South African Machine Solves Acute Compaction Problem. Construction in Southern Africa, May 1970, pp. 31-35.
7. van Vuuren, D. J. Rapid Determination of CBRs With the Portable Dynamic Cone Penetrometer. The Rhodesian Engineer, Vol. 7, No. 5, Sept. 1969, pp. 852-854.
8. Burmister, D. M. The Importance and Practical Use of Relative Density in Soil Mechanics. Proc. Am. Soc. for Testing and Materials, Vol. 48, 1948, pp. 1249-1268.
9. Akroyd, T. N. W. Laboratory Testing in Soil Engineering. Soil Mechanics Ltd., London, 1957.
10. Knight, K. The Collapse of Structure of Sandy Subsoils on Wetting. PhD thesis, University of the Witwatersrand, Johannesburg, Transvaal, 1962.
11. Foster, C. R., and Ahlvin, R. G. Compaction Requirements for Flexible Pavements. HRB Bull. 289, 1961, pp. 1-21.

PREDICTION OF THE BEHAVIOR OF ELASTOPLASTIC ROADS DURING REPEATED ROLLING USING THE MECHANO-LATTICE ANALOGY

W. O. Yandell, School of Highway Engineering, University of New South Wales, Sydney

Repeated axial load triaxial tests on a road material are described. They showed that the plasticity of an elastoplastic road material decreases more with repeated loading than with continuous loading. When the cell pressure is high the damping factor and the elastic compliances also decrease more rapidly. The initial elastoplastic characteristics of a road material were ascribed to a hypothetical pavement being repeatedly traversed by a rigid roller and by a pneumatic tire respectively. The author's mechano-lattice analogy was used to calculate the transient and residual stress patterns after each of six passes. The distribution of horizontal flow of the hypothetical pavement was calculated by the same means. These theoretical findings compared qualitatively with the results of corresponding laboratory tests. It was shown that a maximum residual stress is attained, in this case, after six rolling passes and that the road material flows forward against the direction of traffic movement in a complicated but predictable manner. It is thought that the flow behavior may contribute to the growth of uneveness on road surfaces.

•APART from the author's previous work (1, 2, 3), some useful predictions of subgrade stress-strain behavior have been made. The predictions are for stationary pavement loads only, and they fall into two classes: (a) when the subgrade material is able to consolidate on a temporal basis and (b) where the subgrade is assumed to obey the linearized theory of elasticity. The assumptions that were made, however, rendered both classes unrealistic in view of the large nonelastic components of strain in pavements and subgrades, particularly under construction and fluctuating moisture conditions. The effect of stress-strain hysteresis is also neglected.

This paper describes the author's triaxial tests (2) involving the measuring of the damping and stress-strain properties of a road material subjected to a large number of repeated axial loads. It is shown how these results can be used with the author's mechano-lattice analogy to predict the instantaneous and residual stress and plastic strain behavior in a hypothetical pavement subjected to repeated rolling.

REPEATED-LOAD TRIAXIAL TESTS

Apparatus

It is difficult to produce in a test sample the precise stress-strain regime that a road material is likely to sustain under the action of passing traffic. Triaxial conditions, rotating axes of principal stress, and cyclically varying major, minor, and intermediate principal stresses all must be capable of being controlled and varied in a realistic laboratory material test. In the tests described here and reported earlier (4, 5), a compromise was made between instrumentation difficulties and reality of test conditions.

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A triaxial testing machine, in which the axial load could be applied and removed with a constant rate of strain once a minute, was used. The adjustable cell pressure was held constant during a test involving 10,000 load applications. A number of sequences of readings, recorded by automatic photography, were distributed on a logarithmic basis throughout the period of a test. The cyclic and permanent axial and volumetric deflections and corresponding fluctuating loads were recorded. Stress-strain hysteresis loops were plotted from each sequence of readings, and a damping factor equal to the area in the loop divided by the area under the loading curve was calculated.

Tests

Tests are described to demonstrate how repeated loading can change the stress-strain characteristics of a road material. A 2 x 2 x 3 factorial experiment was carried out on a mixture of 50 percent kaolin and 50 percent sand with a moisture content of 12.4 percent. An X-ray diffractogram of the kaolin revealed 20 percent ground quartz, 50 percent kaolinite, 5 percent illite, 15 percent mixed layer minerals, and 10 percent montmorillonite. Two levels of cell pressure (5 psi and 40 psi), two levels of saturation (dry densities of 118 and 124.6 pcf—98 percent saturated), and three levels of repeated axial stress (52, 62, and 72 percent of the ultimate strength) were used. The ultimate strength was taken as the stress at 18 percent axial strain. A nuclear method was used to check the uniformity of compaction of the samples.

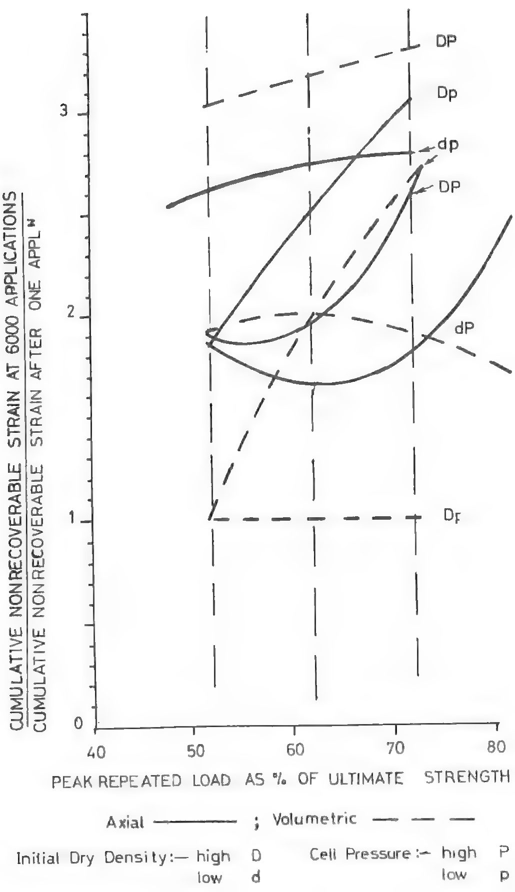


Figure 1. Ratio of cumulative plastic strain after 6,000 load applications over that after one application versus the magnitude of repeated loads.

Results

Figure 1 shows the ratio of the cumulative plastic strain after 6,000 cycles of load over that after one cycle versus the magnitude of the repeated load. It can be seen that axial plastic deformation increased with repeated loading, especially when the cell pressure was low. However, Figure 2 shows the typical manner in which the rate of increase in cumulative plastic deformation gradually decreased until it almost ceased at a high number of load cycles. This demonstrates how a subgrade becomes more stable with the number of load applications or with time when loaded. Creep tests (4) have shown that repeating the load is not very much more effective than holding the load constant in effecting this gradual reduction in plasticity. The saturation increased during the tests, as is evident from the increase in dry density.

The Young's chord moduli of the samples under low cell pressure did not increase greatly with repeated loading, as is shown in Figure 3. The increase in rigidity when the cell pressure was high resulted from the greater consolidation of the samples under high hydrostatic pressures.

The damping factor influences rolling resistance and, when the hysteresis loops are open, can indicate plastic deformation. As shown in Figure 4, it decreased with repeated loading; more so when the cell pressure was high. The increased consolidation under high cell pressures may

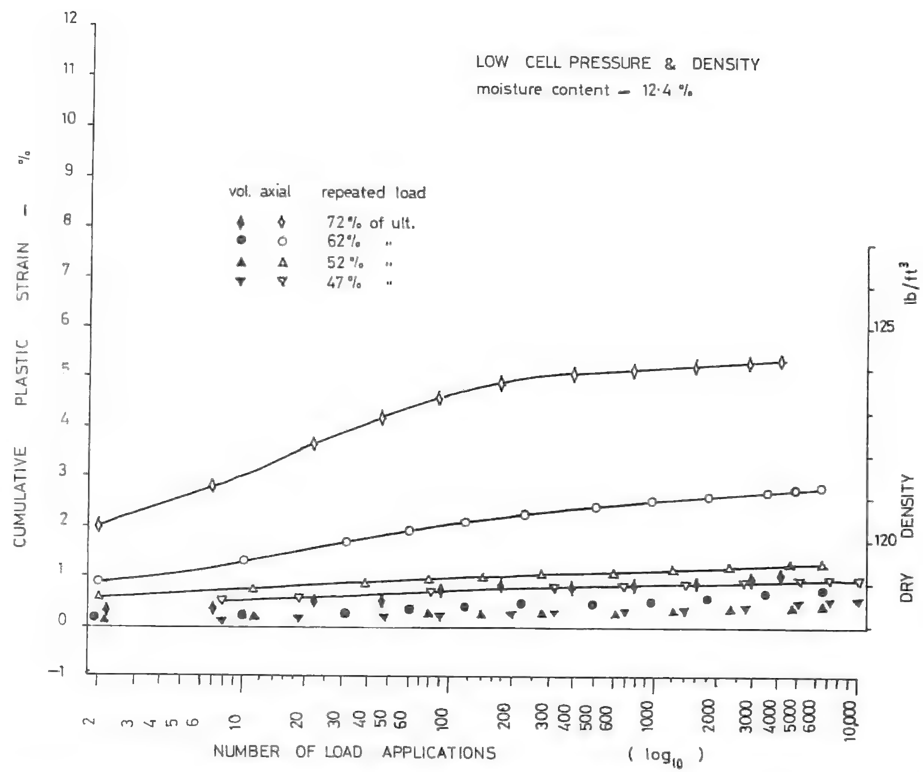


Figure 2. Cumulative plastic strain versus number of load applications applied to the unsaturated sample with a cell pressure of 5 psi.

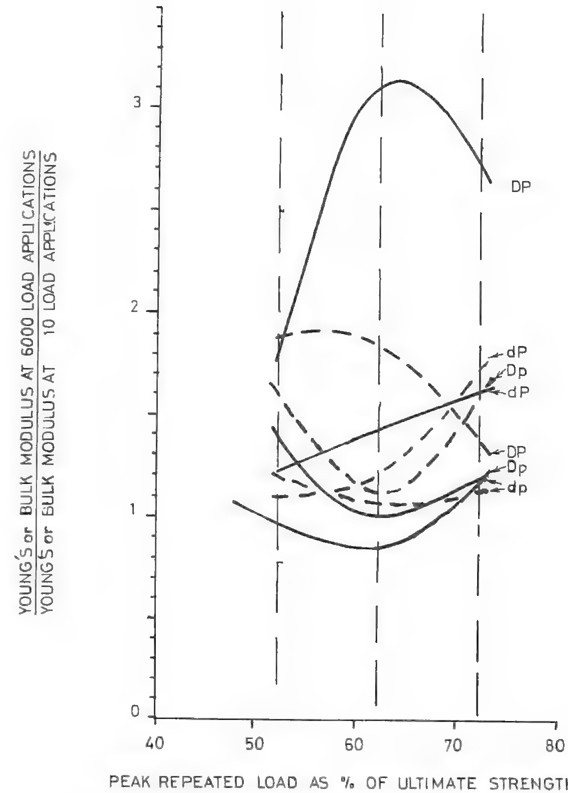


Figure 3. Ratio of the Young's or bulk modulus after 6,000 load applications over that after 10 applications versus the magnitude of repeated loads.

have produced more elastic bonds and hence a lower damping factor.

The author has shown (4) that the damping properties are a function of the average internal friction of the soil. This friction could stem from the bond making and braking, from pore fluid flow, or from the "Coulomb friction" of grain rubbing grain. It has been subsequently shown (6) that sand grains in the test soil do break up after a large number of repeated loadings presumably because of the grains rubbing together.

When the moisture content of a sample was gradually increased from 12 to 13 percent during a repeated loading test, the Young's modulus decreased and the damping factor and plastic strain increased. Some of the hysteresis loops are shown in Figures 5 and 6.

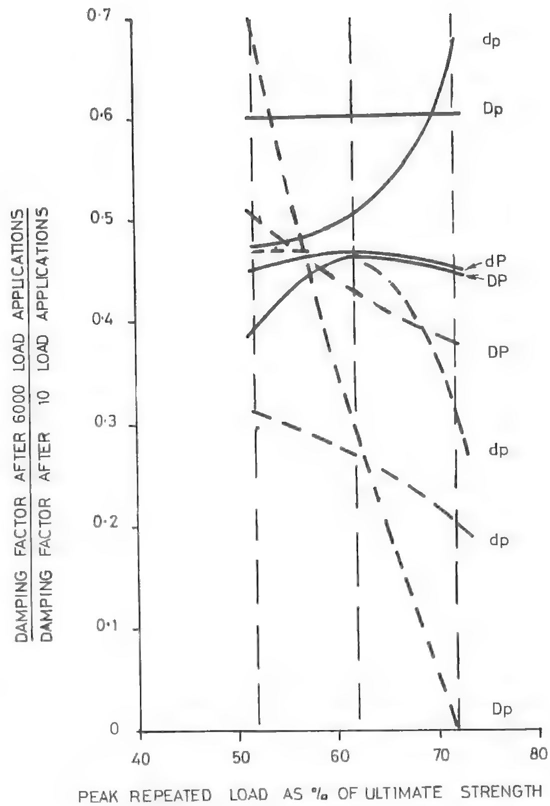


Figure 4. Ratio of the damping factor after 6,000 load applications over that after 10 applications versus the magnitude of repeated loads.

SEPARATION OF PLASTIC BEHAVIOR

Figure 7 shows the axial stress-strain hysteresis loops at the 17th and 10,309th application of load to an unsaturated sample with a low cell pressure (the volumetric behavior is shown in Figure 8). The commencement of each loop is reoriented to a common point to facilitate comparison.

It will be noted that even the loop for the 17th load application shows a large plastic strain. In fact, part of the area of the loop (which is large on the first load cycle and decreases with subsequent cycles) can be ascribed to plastic deformation. Figure 9 shows a typical and realistic hysteresis loop from an elastoplastic soil containing no elastic hysteresis and in which the change in plastic strain per unit absolute change in instantaneous stress is a function of the absolute instantaneous cycling stress. A loop containing the elastic as well as the plastic damping energy is shown by the broken line in Figure 10.

A theoretical method will now be described to demonstrate the elastoplastic behavior of a soil comprising a repeatedly rolled pavement. For clarity the characteristics of the material were assumed, contrary to the preceding test findings, to remain as they were on the first load cycles.

THE MECHANO-LATTICE ANALOGY

Elastoplastic Behavior

The author's mechano-lattice analogy described elsewhere (1, 2, 3) was used for the prediction of stress-strain behavior of an elastoplastic soil in a hypothetical pavement being subjected to repeated rolling. Only the elastoplastic components, as in the soil described previously, were used and were considered to have constant characteristics throughout the repeated rolling. For clarity, the elasto-frictional components (1) were ignored.

Figure 10 shows the elastoplastic components of the axial load versus deflection hysteresis loop recorded during a typical first or second load cycle. It will be noted that

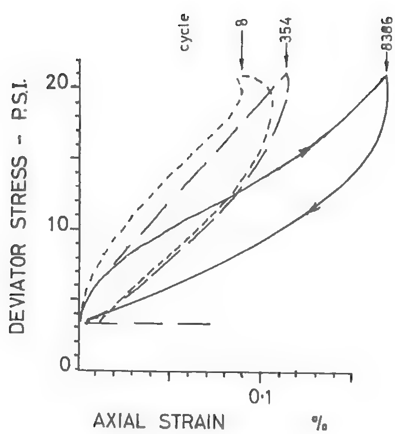


Figure 5. Axial stress-strain hysteresis loops for the 8th, 354th, and 8,386th cycles of a 47 percent of ultimate load on the unsaturated sample with a cell pressure of 5 psi as the moisture content increased from 12 to 13 percent, a dry density of 118 lb/ft^3, and a 47 percent of ultimate load.

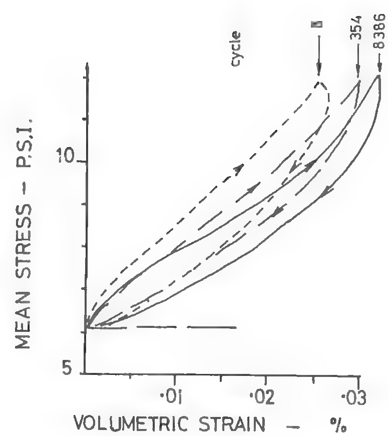


Figure 6. Volumetric stress-strain hysteresis loops for the 8th, 354th, and 8,386th cycles of a 47 percent of ultimate load on the unsaturated sample with a cell pressure of 5 psi as the moisture content increased from 12 to 13 percent.

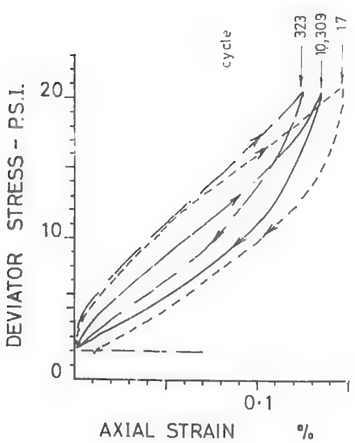


Figure 7. Axial stress-strain hysteresis loops for the 17th and 10,309th cycles of a 47 percent of ultimate load on the unsaturated sample with a cell pressure of 5 psi, a moisture content of 12.4 percent, and a dry density of 118 lb/ft^3.

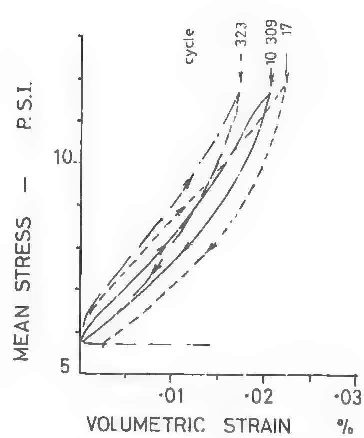


Figure 8. Volumetric stress-strain hysteresis loops for the 17th and 10,309th cycles of a 47 percent of ultimate load on the unsaturated sample with a cell pressure of 5 psi.

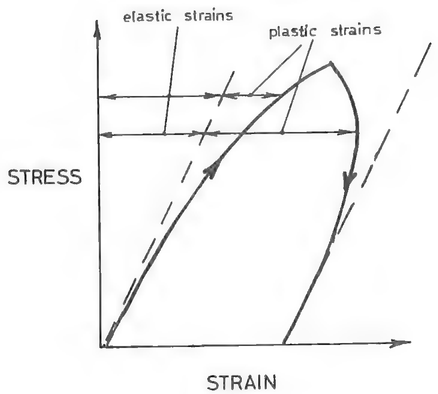
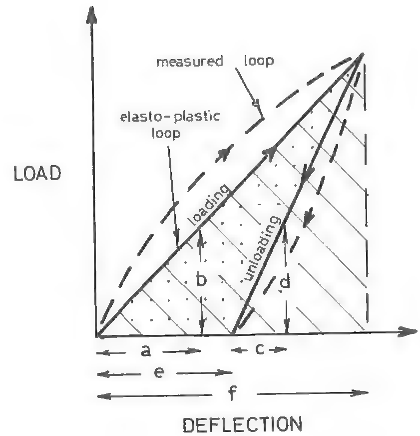


Figure 9. The axial stress-strain hysteresis loop of a hypothetical elastoplastic soil.



$E'_L = b/a \qquad E'_U = d/c$

Figure 10. The simplification and partition of an axial elastoplastic load-deflection hysteresis loop from a measured loop.

a further simplification was made beyond that shown in Figure 9. The change in plastic deformation per unit absolute change in instantaneous load was assumed to be independent of instantaneous load and proportional instead to the mean absolute cycling load for one cycle. This produced a scalene-triangel shaped open loop that is more mathematically tractable.

The axial behavior can be considered as that of an elastic element that has a greater modulus when unloading, E_U, than when loading, E_L (Fig. 10). The plastic factor is e/f and is equal to the damping factor, ξ, which is the dotted area in the loop divided by the hatched area under the loading line.

Similar behavior will be ascribed to individual elements making up each two-dimensional unit of the analogy. The analogy simulates a long section of a pavement suffering plane stress (Fig. 11).

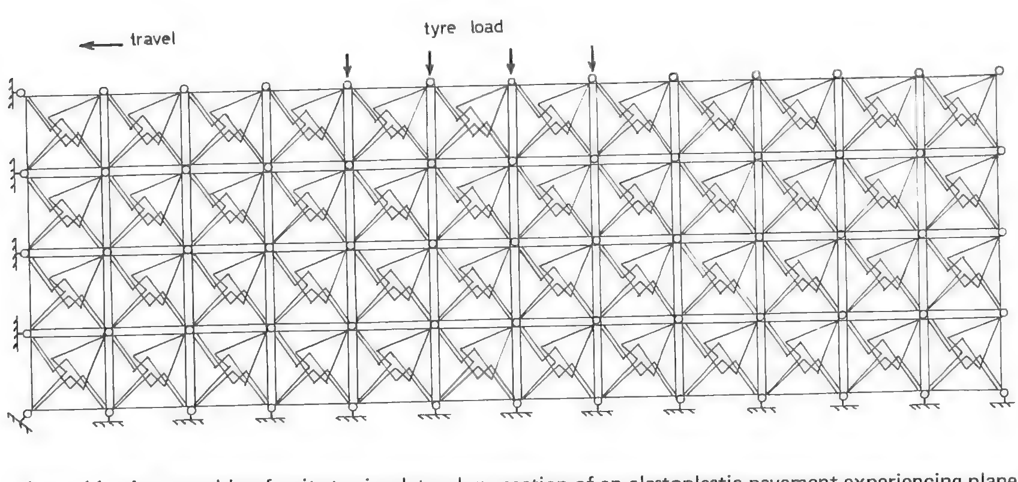


Figure 11. An assembly of units to simulate a long section of an elastoplastic pavement experiencing plane stress.

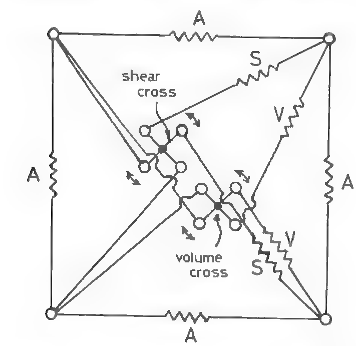


Figure 12. A unit of the lattice structure simulating the behavior of the soil.

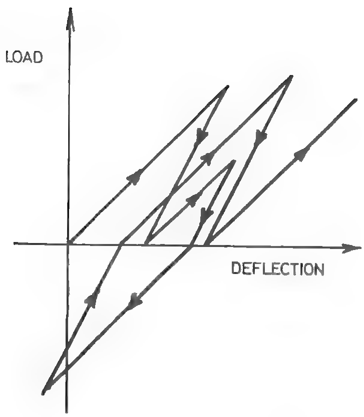


Figure 13. Possible load-deflection behavior of an element or model.

Plane Stress Simulation Unit of the Analogy

A diagram of the model unit is shown in Figure 12. The elastic elements exhibit one stiffness when the absolute load is increasing and a higher stiffness when the absolute load is decreasing. This produces plastic deformations that are not time-dependent. A possible load-deflection path exhibited by an element is shown in Figure 13. When a positive or negative unloading path crosses the deflection axis, it becomes a loading path and hence its slope decreases as shown.

Figure 10 shows the load-deflection path of one positive load cycle on any element. The slope of the loading path is equal to the loading stiffness coefficient, E'_L, of the element. The slope of the unloading path is E'_U. The damping energy per cycle is represented by the area in the loop. The damping factor is the area in the loop divided by the hatched area. The plastic factor is the plastic strain divided by the total strain.

The model shown in Figure 12 simulates the behavior of a material with a particular rigidity, plasticity, and Poisson's ratio. The two crosses, free to rotate, enable shear and volumetric behavior to be independent and thus allow various Poisson's ratios to be simulated. The left-hand cross permits the shear elements to be activated only when shear deformation is taking place. The right-hand cross permits the volume elements to be activated only during volume changes.

The stiffness coefficients of the elements are calculated by frame analysis for the representation of a non-buckling plate of unit thickness (7):

$$E'(\text{L or U}) = \frac{E(_{\text{L}}\text{ or }_{\text{U}})D}{2(1+\sigma)} \quad \text{(horizontal and vertical elements)}$$

$$= \frac{E(_{\text{L}}\text{ or }_{\text{U}})D}{(a+\sigma)\sqrt{2}} \quad \text{(the two shear elements)}$$

$$= \frac{E(_{\text{L}}\text{ or }_{\text{U}})D}{1-\sigma^2} \quad \text{(the two volume elements)}$$

where

E(L or U) = Young's loading or unloading modulus
D = The length of a side of the model
σ = Poisson's ratio.

The complex case of a roller moving on an elastoplastic material is solved by connecting a number of models (Fig. 12) at their joints, as shown in Figure 11 to represent

a long section of road. The left-hand and base joints are fixed, and the right-hand and upper joints are free to move. A drum roller effect is represented by displacements to the central upper surface joints conforming to its radius. A pneumatic tire effect is produced by placing fixed loads on a number of the central upper joints.

SOLUTIONS

Solutions are achieved by an iterative procedure and by using an electronic digital computer. The program flow chart is shown in Figure 14. The force history of each element is traced by its consecutive changes in length and by its load. At the beginning of each calculation cycle the lengths of all elements, measured from joint to joint, are computed.

Stages in the cycles of load-deflection that an element experiences as the wheel rolls from right to left are represented by the states of corresponding elements examined in turn while moving from left to right in the same row. Thus the load-deflection path or hysteresis loop of each element is started from fixed values on the left and traced from one unit to the next by comparing compressions and elongations.

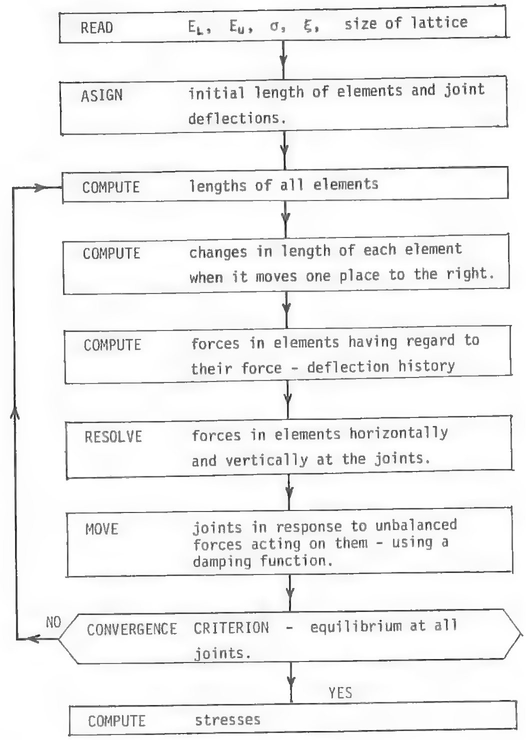


Figure 14. Flow diagram of the computer program.

If one element is shorter than the corresponding element in the model on the left, an increase in compressive loading is indicated when both elements have compressive loads. The change in length is multiplied by the element's stiffness coefficient, E_L', to obtain the increase in load. The change in load in the element is added to its original load. When a load-deflection path crosses a line of zero load as the absolute load is decreasing, the stiffness coefficient is changed from E_U' to E_L'. Thus closed hysteresis loops can occur if the load is reversed. The foregoing process of computing the forces in all the elements is repeated for each element while moving along the rows.

The forces in the elements are next resolved horizontally and vertically at the adjacent joints. Each joint is then moved in the direction of those unbalanced forces by an amount that is a function of them.

This process of calculating lengths, changes in length, forces in elements, and movements of joints is repeated until convergence at about 150 cycles for 120 joints. Upon convergence, when the joints are in equilibrium, the stresses at the center of each model are calculated by averaging and resolving the forces in the appropriate elements.

For the solution of the second pass, the rectilinear and horizontally homogeneous residual stresses and strains following the first pass of the roller or wheel are ascribed to all the material at the start of second-pass calculations. In this way any number of repeated rollings can be simulated. In the present example only six passes were needed to achieve a constant residual stress distribution.

RESULTS

A hypothetical elastoplastic pavement 15 in. long and 2 in. thick resting on a rigid base was analyzed. It was, in simulation, repeatedly rolled by (a) a 40-in. diameter rigid roller and (b) a pneumatic tire. A loading modulus, E_L, of 5,000 psi; an unloading

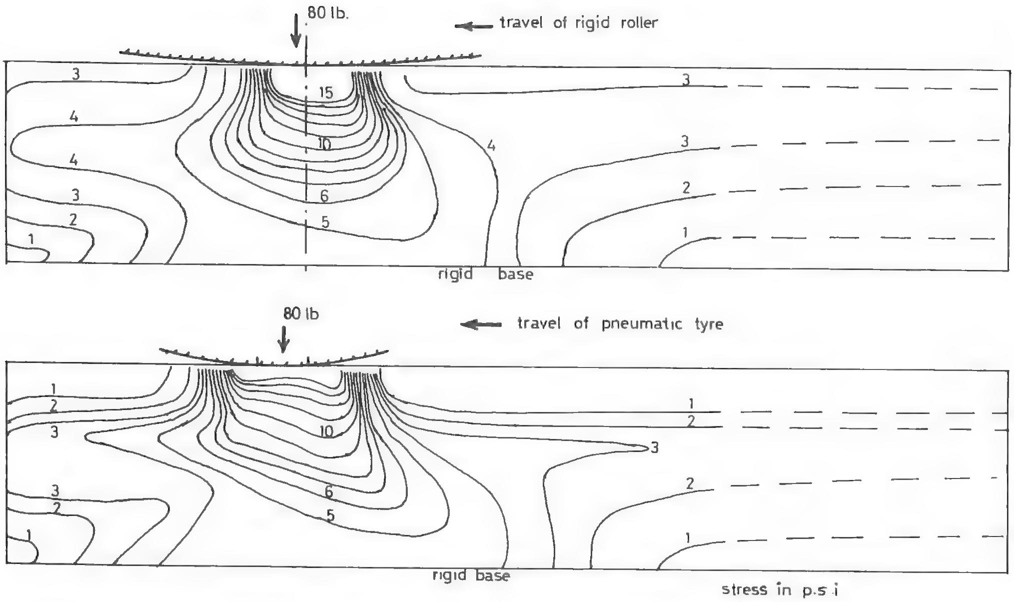


Figure 15. Transient and residual maximum shear stress patterns calculated during the 6th pass of the rigid roller and of the pneumatic tire.

modulus, E_U, of 10,000 psi; a Poisson's ratio, σ, of 0.4; and a damping and plastic factor of 0.5 were assumed.

Residual Stresses

Figure 15 shows the maximum shear stress distributions involved by the sixth passes of the roller and of the pneumatic tire respectively. The sum of the transient stresses, which moved with the load, and the residual stresses is shown in full lines. The residual stresses that remained are shown by broken lines. Although the loads were similar, the roller induced more intense stresses, especially forward of the moving load.

The buildup of residual stresses during repeated rolling is one of the most important properties of elastoplastic road materials. In the present problem the horizontal residual stresses increased at a reducing rate until they became constant after the sixth pass (Fig. 16).

The distribution with depth of the horizontal and vertical residual stresses reresulting from the repeated passing of the rigid roller and the pneumatic tire are shown in Figure 17. In each case the vertical residual stress was small and tensile and changed little with the number of passes. The vertical residual tensile stress would be partly offset by the neglected weight of the road material.

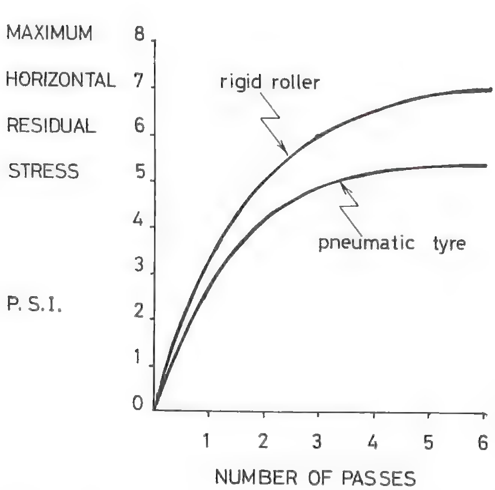


Figure 16. Maximum horizontal residual stresses versus number of passes.

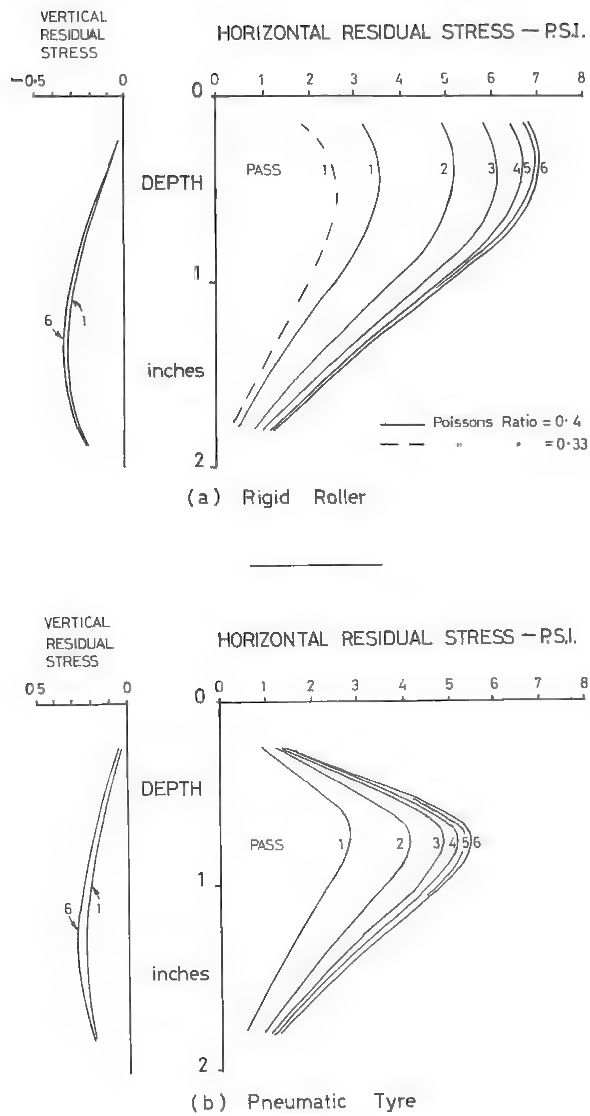


Figure 17. Residual stress distribution with depth.

The roller caused the maximum horizontal residual stress to build up close to the surface, whereas the pneumatic tire caused this maximum to occur well below the surface, leaving the surface unstressed. The rigid roller induced greater residual stresses. Increasing the Poisson's ratio increased the residual stresses; i.e., greater residual stresses would occur in well-compacted or highly saturated materials if other factors are constant.

The rolling resistance of the rigid roller was calculated from the torque produced by the nonsymmetrical reactions at the surface of contact—that of the tire from the uphill slope of the surface of contact. In both cases the rolling resistance decreased at a low rate as passes continued. The roller had the greater resistance to rolling (Fig. 18).

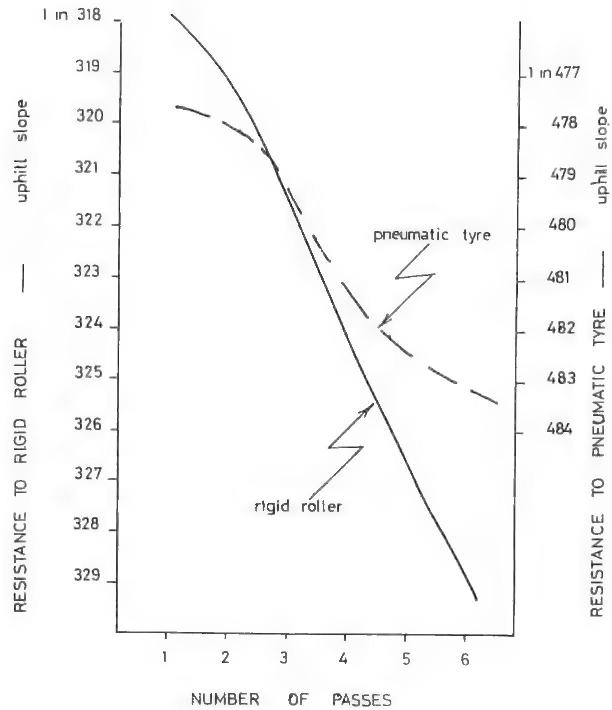


Figure 18. Rolling resistance.

Cumulative Strains

If this hypothetical road material were to lose its plastic hysteresis after a large number of passes—as it usually does in practice—the surface deformation would become symmetrical, and the pavement would lose its resistance to rolling because no elastic hysteresis was present. The vertical plastic and elastic strains per pass remained constant.

The horizontal flow that varied with depth caused complex shear behavior. There was a marked difference in shear behavior between that induced by the roller and that induced by the pneumatic tire, as is evident in Figure 19, which shows the cumulative horizontal flow at each depth.

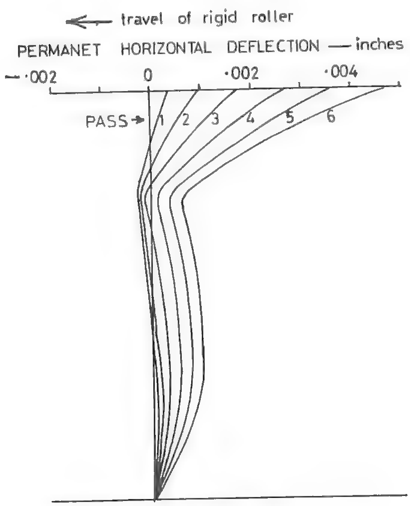
The roller induced a very marked forward permanent shear in the top layer that continued with the number of passes. The lower layer also sheared forward, whereas the intermediate layers sheared slightly backward. The whole body of the soil moved steadily forward (against the direction of rolling). The phenomenon of forward shear was shown by Hamilton (8) to exist in repeatedly rolled copper. Merwin and Johnson (9) used approximate estimates based on the Hertz distribution to check Hamilton's findings.

In contrast the simulated pneumatic tire induced a backward shear in the top layer, while the deeper soil sheared forward. The body of material moved forward at a greater rate than when the similarly loaded roller was passing.

Whether the greater forward shear associated with rigid rolling causes disruption at the surface or aids in compaction must depend on the nature of the road material.

QUALITATIVE VERIFICATION OF THE THEORETICAL FLOW PREDICTIONS

The theoretically predicted cumulative deformational behavior of the hypothetical elastoplastic road material was qualitatively checked with practice. This was done by



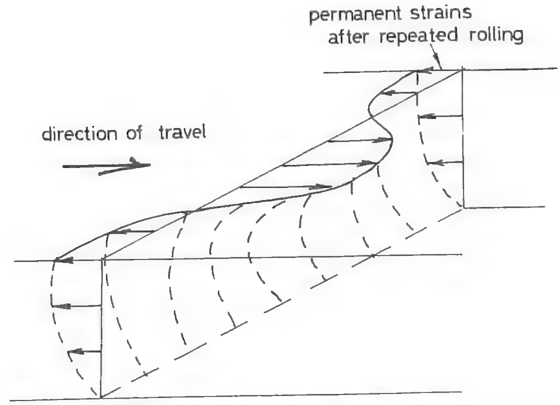


Figure 20. The measured and estimated horizontal flow pattern in Sparks and Davis test track.

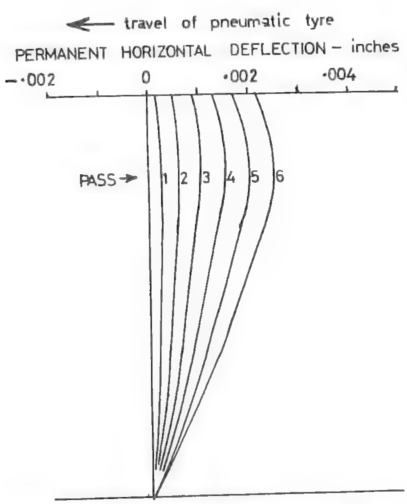


Figure 19. Horizontal flow of material at each depth.

repeatedly rolling a 2-in. diameter roller on a two-colored layer of modeling clay $\frac{1}{2}$ in. thick. A vertical plane was formed at the interface of the two colors so that flows could be observed.

When the roller was torque driven, as with the theoretical examples, the flow was against the direction of rolling as predicted. When the roller was moved by a horizontal force applied at the axle, the flow was against the direction of travel in the body of material but the surface layer was moved differentially back in the direction of the roller travel.

Subsequent to the work described here, Sparks and Davis (10) used a loaded, free-running pneumatic tire repeatedly rolling in the one direction on the central third of a long straight test track. The 6-in. thick test pavement consisted of $\frac{3}{16}$ in. to dust size crushed dolerite, 10 percent kaolinite binder, and a moisture content of $5\frac{1}{2}$ percent. After a large number of passes, the surface of that part of the track rolled by the tire moved in the direction of travel, while the outer nontraversed parts moved against the direction of travel, as shown by the full line in Figure 20. The shape of an initially vertical plane is shown by broken lines. Its subsurface shape is this author's estimate based on the foregoing theoretical work and the laboratory tests with modeling clay.

CONCLUSION

The repeated load soil tests described here have shed light on energy and flow characteristics of road materials. At least one type of material undergoes great changes in damping factor, stiffness, and plasticity while being either repeatedly or constantly loaded. It was shown that a small change in moisture content can drastically change the values of the foregoing characteristics from safe levels.

The theoretical analysis and confirming practical proof described assumed that the values of the material stiffness and plastic and damping characteristics of a pavement

were constant throughout repeated traverses by loaded wheels or rollers. It was shown that the pavement material flows in the opposite direction to the traffic movement. The differential of the distribution of the material flow with depth may lead to some type of surface contortion and hence to failure. When the road material properties vary from location to location, the changes in flow rate may lead to the surface uneveness that renders a road unservicable. The theoretically demonstrated horizontal residual stresses could lead to buckling and perhaps corrugations of the surface.

As shown by the repeated-load triaxial tests on soil, plasticity almost disappears after a large number of load applications; in this case the flow and associated contortion of the road surface comprising the soil would cease to worsen. In the case of the earth pavement, however, an increase in moisture content can, as previously shown, reestablish plastic flow. In the case of bitumen roads, an increase in temperature can also reestablish plastic flows and further surface irregularities would appear.

ACKNOWLEDGMENT

The author wishes to thank Professor D. F. Orchard for his encouragement.

REFERENCES

1. Yandell, W. O. Stress Distribution Associated With Rolling Resistance. Australian Road Research, Vol. 3, No. 1, March 1967, pp. 3-13.
2. Yandell, W. O. Rolling Resistance and Stress Distribution Associated With Plastic Hysteresis. Inst. of Engrs. Aust. Jour. Paper No. 2370, Oct.-Nov. 1967, p. 179.
3. Yandell, W. O. The Effect of Repeated Rolling of Elastoplastic Roads. Australian Road Research, Vol. 3, No. 7, Sept. 1968, pp. 3-13.
4. Yandell, W. O. The Effect of Repeated Loading on Road Making Materials. M.E. thesis, University of New South Wales, 1966.
5. Yandell, W.O. Some Effects of Repeated Loads. Proc. Third Conf. of Australian Road Research Board, Sydney, Sept. 4-5, 1966. A.R.R.B., Vol. 3, Part 2, (paper 279), 1967, pp. 1100-17.
6. Shackel, B. Private communication, 1970.
7. McHenry, D. A Lattice Analogy for the Solution of Stress Problems. Jour. I.C.E., Vol. 21, No. 2, (paper 5350) Dec 1943, pp. 59-82.
8. Hamilton, G. M. Plastic Flow in Rollers Loaded Beyond the Yield Point. Proc. Inst. Mech. Engr., Vol. 177, No. 25, (Appl. Mech. Group Paper), 1963, pp. 667-75.
9. Merwin, J. E., and Johnson, K. L. An Analysis of Plastic Deformation in Rolling Contact. Proc. Inst. Mech. Engr., Vol. 177, No. 25, (Appl. Mech. Group Paper), 1963, pp. 676-85.
10. Sparks, G. H., and Davis, E. H. First Road Base Experiment With a Laboratory Test Track. A.R.R.B., 5th Conf., August 1970.

A GENERALIZED INVESTIGATION OF POTENTIALLY POOR SOIL SUPPORT BY REGIONAL GEOMORPHIC UNITS WITHIN THE CONTERMINOUS 48 STATES

Matthew W. Witczak, The Asphalt Institute; and
C. W. Lovell, Jr., and E. J. Yoder, Department of Civil Engineering, Purdue University

This report qualitatively assesses the potential for poor soil support within the conterminous 48 states. For each of 97 physiographic sections, an estimate was made of (a) the frequency of occurrence of organic deposits and (b) the combined frequency of occurrence-severity rating of clayey deposits. In order to assess the regional character of the poor support problem, a national soil textural map was developed. Frequency of occurrence ratings for organic deposits were determined directly from this map for each section. For clayey deposits, a severity scale, based on a relationship between the soil texture and the Unified Soil Classification System, was combined with the frequency of occurrence rating of clayey deposits obtained from the national soils map. The study indicated the limited regional distribution of organic terrain in the 48 states. This distribution is concentrated in youthful (geomorphic) glacial and coastal terrain due to the rather poorly integrated drainage system often associated with these regional geomorphic areas. Although clayey deposits occur throughout the 48 states, most are found east of the Rocky Mountains because the climatic, topographic, and parent material factors are generally more favorable there.

•ALTHOUGH engineering design and construction decisions are unique solutions to specfic problems, these decisions may depend strongly on the store of highly relevant experiences that engineers use as background or perspective input. The requisite input for highway design and construction decisions is developed through a process of convergence, for example, by moving from a general understanding of a large piece of geography to the specifics of a site or route that is no more than a point or a thin line on any but a very large-scale map.

The search for geographic units that demonstrate significant homogeneity in ground conditions, other environmental factors, engineering problems, and design and construction practice has led a number of engineers to study the work of physiographers and regional geomorphologists. These scientists classify and map areas on the basis of their mode of topographic expression, which in turn depends principally on the factors of structure, process, and stage. Because these factors can be practically interpreted as parent material, origin, and age (19), it is not surprising that physiographic mapping is useful. The mapped units of interest to engineers are the province, the section, and the subsection.

The objective of this paper is to report an investigation of the distribution of soils that afford potentially poor support for highway structures and the correlation of this distribution to an extant system for the conterminous 48 states. Two general soil categories were identified as providing "poor" support: organics and clays. The organics are extremely poor as foundations for embankments or other structures and are, of course, unacceptable as subgrade or embankment materials (except in certain non-

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critical portions of the embankment). The clays are potentially troublesome as foundations or in compacted layers because of their low permeability and their sensitivity to changes in water content, which cause changes in strength and volume.

The classification selected was a slightly modified version of the Woods-Lovell Engineering-Physiographic System presented in 1960 (17), which has 97 "unique" areas (physiographic sections). Table 1 lists each section and provides a code for its location shown in Figure 1.

METHOD OF ANALYSIS

The qualitative evaluation of potentially poor soil support areas within each section was based primarily on a national soil texture map, developed by the senior author and used in conjunction with the map of physiographic sections. For the clayey soils a combined severity-frequency rating was devised, whereas for the organics ratings were based on the relative frequency of occurrence.

The following portions of this paper briefly describe the methods used (a) to develop the soils texture map and (b) to obtain an estimate of the magnitude of the poor soil support problem within each of the 97 sections composing the conterminous 48 states.

Soil Texture Map

The generalized soil textural map of the United States was developed to a scale of 1:2,500,000, which corresponds to the scale of the national geologic map (11) as well as the pedologic map (4).

Many references were consulted in the preparation of the map. Principal among these were reports and mappings of soil distribution for national, regional, and state coverage. The national group included several references (5, 6, 9, 13, 17, 18). Regional soil references were available for the western United States (16), north central region (12), southeast region (14), and the northeastern United States (10). Many individual state soil maps were frequently consulted for soil data. The references varied widely in content and date of preparation; e.g., only old coverage was available for some geographic areas (7, 8), whereas very current information was located for others (3, 15).

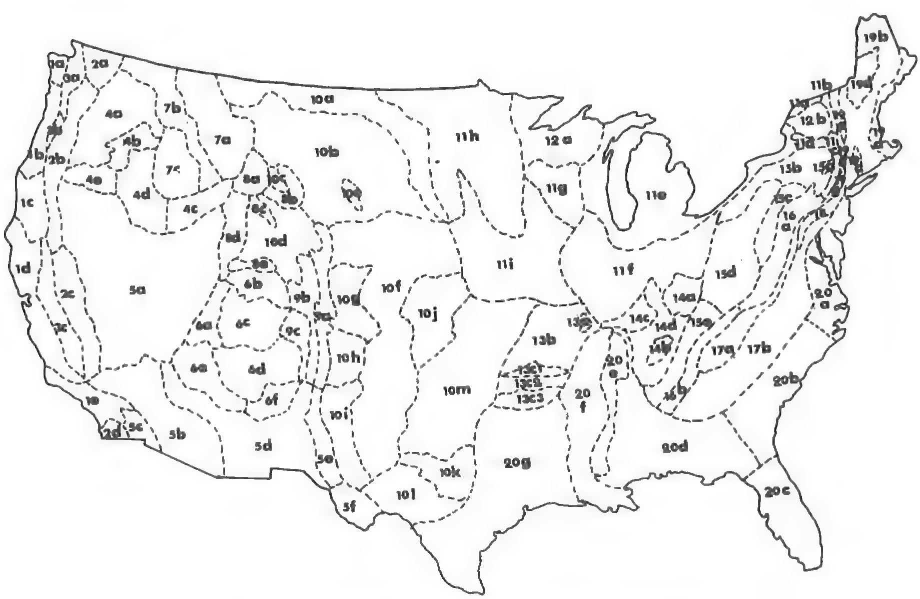


Figure 1. Physiographic diagram of the United States.

TABLE 1

PHYSIOGRAPHIC UNIT CODE

1. Western Mountains of the Pacific Coast Range
   a. Olympic Mountain
   b. Oregon Coast Range
   c. Klamath Mountain
   d. California Coast Range
   e. Los Angeles Range
2. Sierra-Cascade
   a. Northern Cascade Mountain
   b. Southern Cascade Mountain
   c. Sierra Nevada
   d. Lower California
3. Pacific Troughs
   a. Puget Sound
   b. Willamette Valley
   c. California Valley
4. Columbia Plateau
   a. Walla-Walla
   b. Blue Mountain
   c. Snake River Plains
   d. Payette
   e. Harney
5. Basin and Range
   a. Great (Closed) Basin
   b. Sonoran Desert
   c. Salton Trough
   d. Open Basin (Mexican Highland)
   e. Sacramento Highland
   f. Great Bend Highland
6. Colorado Plateau
   a. High Plateaus of Utah
   b. Uinta Basin
   c. Canyon Lands
   d. Navajo
   e. Grand Canyon
   f. Datil
7. Northern Rocky Mountain
   a. Montana
   b. Bitteroot
   c. Salmon River
8. Middle Rocky Mountain
   a. Yellowstone
   b. Bighorn Mountain
   c. Wind River Mountain
   d. Wasatch
   e. Uinta Mountain
9. Southern Rocky Mountain
   a. Front Range
   b. Western
   c. San Juan Mountain
10. Great Plains
    a. Glaciated Missouri Plateau
    b. Unglaciated Missouri Plateau
    c. Bighorn Basin
    d. Wyoming Basin
    e. Black Hills
    f. High Plains
    g. Colorado Piedmont
    h. Raton Upland
    i. Pecos Valley
    j. Plains Border
    k. Central Texas Mineral
    l. Edwards Plateau
    m. Osage Plains
11. Central and Eastern Lowlands
    a. St. Lawrence Lowland
    b. Champlain Lowland
    c. Hudson River Valley
    d. Mohawk River Valley
    e. Eastern Lakes and Lacustrine
    f. Central Till Plain
    g. Driftless
    h. Western Lakes and Lacustrine
    i. Dissected Loessial and Till Plain
12. Laurentian Upland
    a. Superior Upland
    b. Adirondack
13. Ozark and Ouachita
    a. St. Francois Mountain
    b. Springfield-Salem Plateau
    c1. Boston Mountain
    c2. Arkansas Valley
    c3. Ouachita Mountain
14. Interior Low Plateaus
    a. Blue Grass
    b. Nashville Basin
    c. Shawnee Hills
    d. Highland Rim
15. Appalachian Plateau
    a. Catskill Mountain
    b. New York Glaciated
    c. Allegheny Mountain
    d. Kanahwa
    e. Cumberland
16. Newer Appalachian (Ridge and Valley)
    a. Pennsylvania-Maryland-Virginia
    b. Tennessee
17. Older Appalachian
    a. Blue Ridge
    b. Piedmont
18. Triassic Lowland
19. New England Maritime
    a. Seaboard Lowland
    b. New England Upland
    c. Connecticut Lowland
    d. White Mountain
    e. Green Mountain
    f. Taconic
    g. Reading Prong
20. Atlantic and Gulf Coastal Plain
    a. Embayed
    b. Sea Island
    c. Florida
    d. East Gulf
    e. Mississippi Loessial Upland
    f. Mississippi Alluvial Plain
    g. West Gulf

Note: Numbers represent physiographic provinces, letters represent physiographic sections.

TABLE 2

GENERAL SOIL CLASSIFICATION CORRELATION

General Texture	Texture Type	U.S.C.S.[a] Most Probable	U.S.C.S.[a] Other
Coarse	Gravel	GP;GW	GM;GC
	Sand	SM;SP-SM	SM-SC
	Loamy sand	SM	SC
Moderately coarse	Sandy loam	SM	ML;SC
	Fine sandy loam	SM;ML SM;ML	SC;SM
Medium	Very fine sandy loam	SM;ML	
	Loam	CL	ML;ML-CL
	Silt loam	CL	ML;ML-CL
Moderately fine	Sandy clay loam	CL	SC
	Silt	ML	CL
	Silty clay loam	CL	CH
	Clay loam	CL	CH
Fine	Sandy clay	SC;CL	CL
	Silty clay	CH	CL;MH
	Clay	CH	

[a]Unified Soil Classification System.

TABLE 3

GENERALIZED SEVERITY CATEGORY OF POOR SUPPORT POTENTIAL BY TEXTURAL CLASSIFICATION

Category	Textural Classification	Most Probable U.S.C.S. Category	Other U.S.C.S. Categories
1 (least severe)	Sandy clay	SC;CL	
	Sandy clay loam	CL	SC
2	Silt	ML	CL
	Silt loam	CL	ML;ML-CL
	Loam	CL	ML;ML-CL
3	Clay loam	CL	CH
	Silty clay loam	CL	CH
4 (most severe)	Silty clay	CH	CL
	Clay	CH	CL

The mapping technique was designed to retain as much detailed soil information as practicable; this was accomplished by preserving rather adequate descriptions of soil texture. The information available varied from classification by engineering systems to such generalities as "fine textured" or "moderately coarse." Table 2 was developed from other sources (1, 2) to aid in the requisite interpretations and correlations of descriptions.

Every attempt was made to distinguish and map the general texture of the parent material. Within residual soil areas this was not always possible, and major emphasis was placed on the general texture in the subsoil as well as the weathered parent materials.

Organic Deposits

The term organic deposits refers to peat bogs, muck lands, and associated swamps and tidal marshes. The relative frequency of occurrence of these deposits within each physiographic unit was evaluated directly from the national soils texture map. An arbitrary rating code was devised as follows: (VW) very widespread, (M-W) medium-to-widespread, (L-M) limited-to-medium, (N-L) nonexistent-to-limited, and (NE) nonexistent.

Clayey Deposits

Foundation and subgrade problems with inorganic soils are not confined to clays, but a number of difficulties are correlated with clayeyness. Accordingly, the rating of sections was accomplished by a combined consideration of the clayeyness of the soils and the frequency of occurrence of such soils.

The two coarse categories in Table 2, i. e., coarse and moderately coarse, are considered to afford satisfactory support. The finer textures were grouped into four severity categories, based on the most probable Unified Soil Classification given in Table 2; the four categories are given in Table 3.

Severity ratings were then qualitatively formulated for each section by combining distributive information with the general severity categories of Table 3. The arbitrary rating code used the same five descriptors as were used for the organic deposits.

RESULTS

Soil Texture Map

The soil texture map of the United States is shown in six sheets (Figs. 2 to 7). In general, three broad categories of units are mapped: single-textured units, multiple-

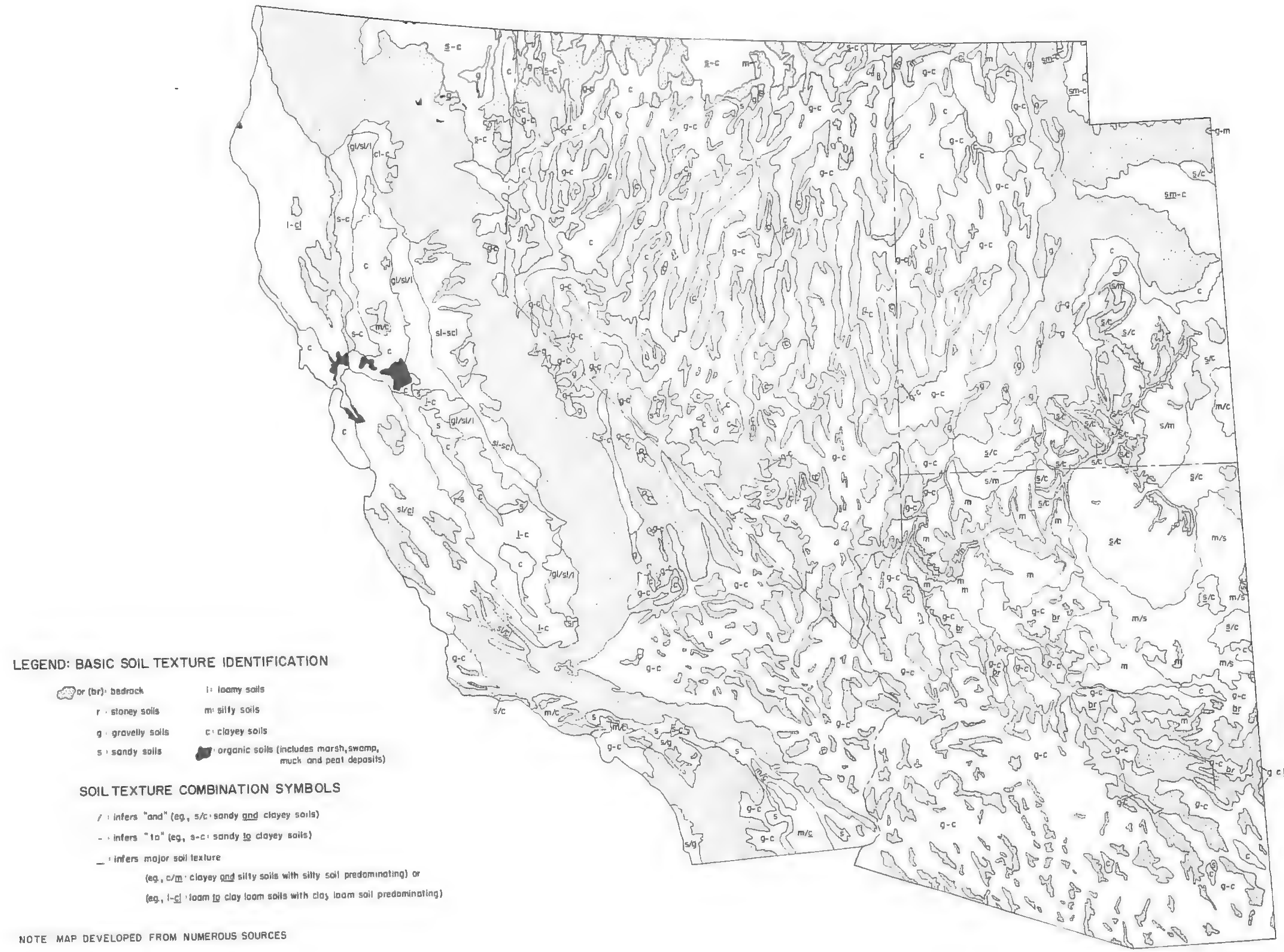
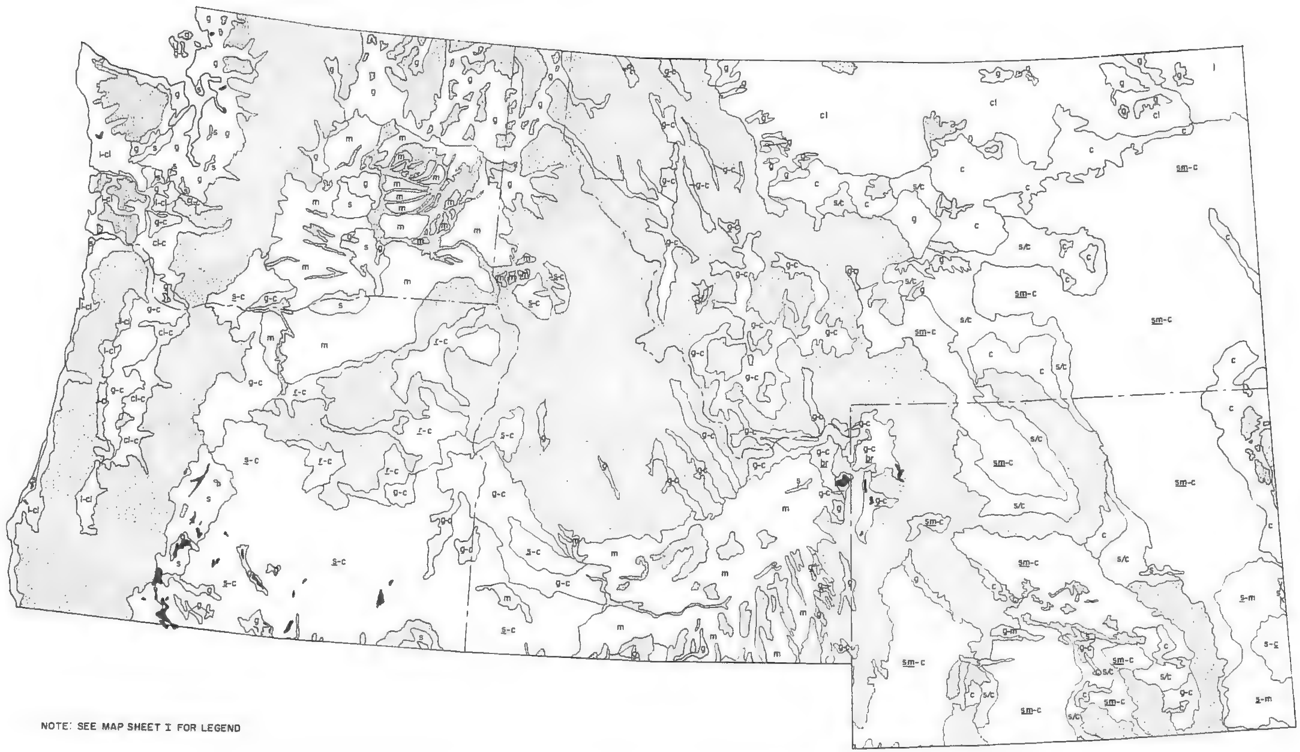


Figure 2. Generalized soil texture map of the continental United States (Map sheet I: southwest United States).



NOTE: SEE MAP SHEET I FOR LEGEND

Figure 3. Generalized soil texture map of the continental United States (Map sheet II: northwest United States).

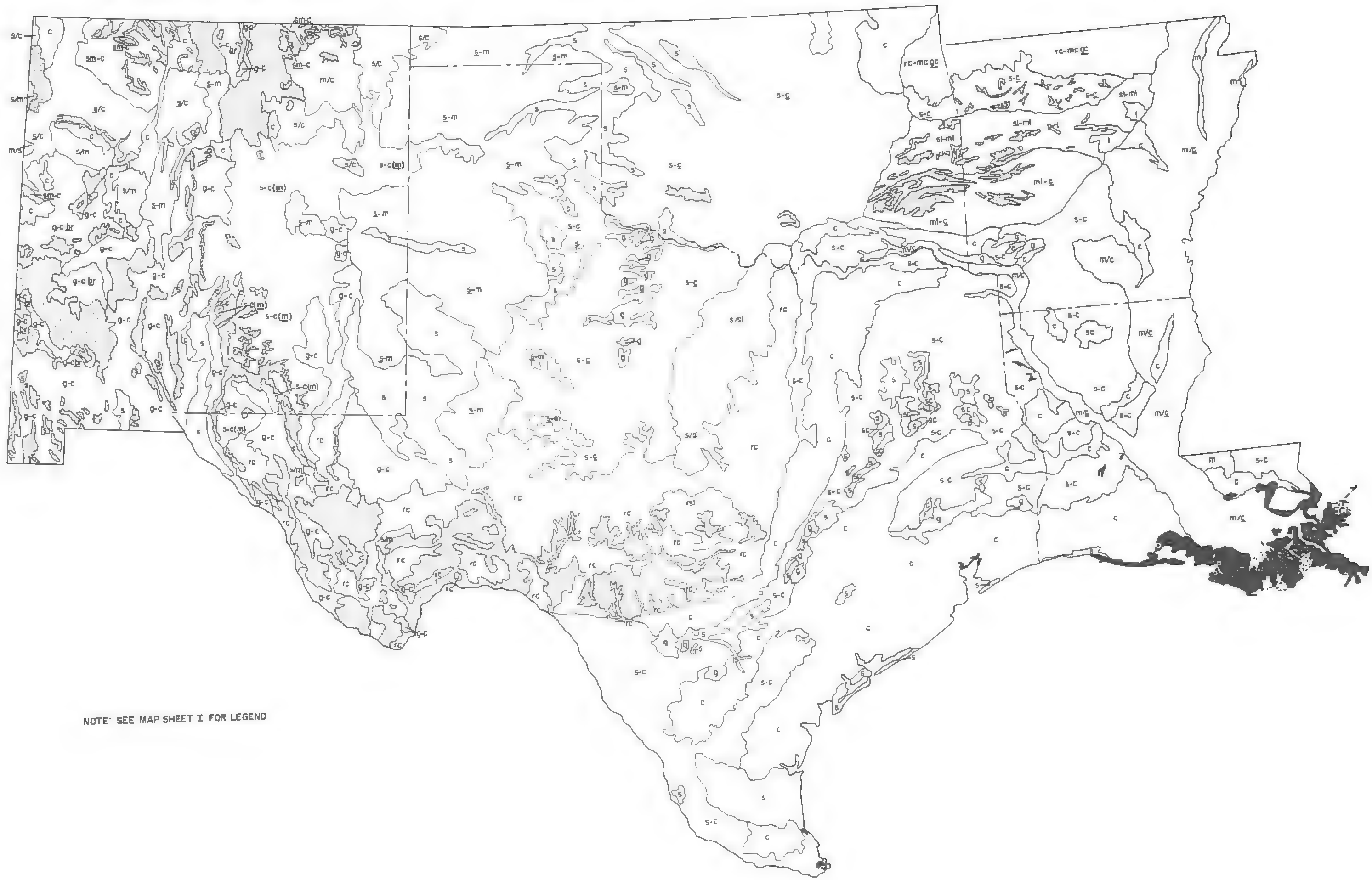
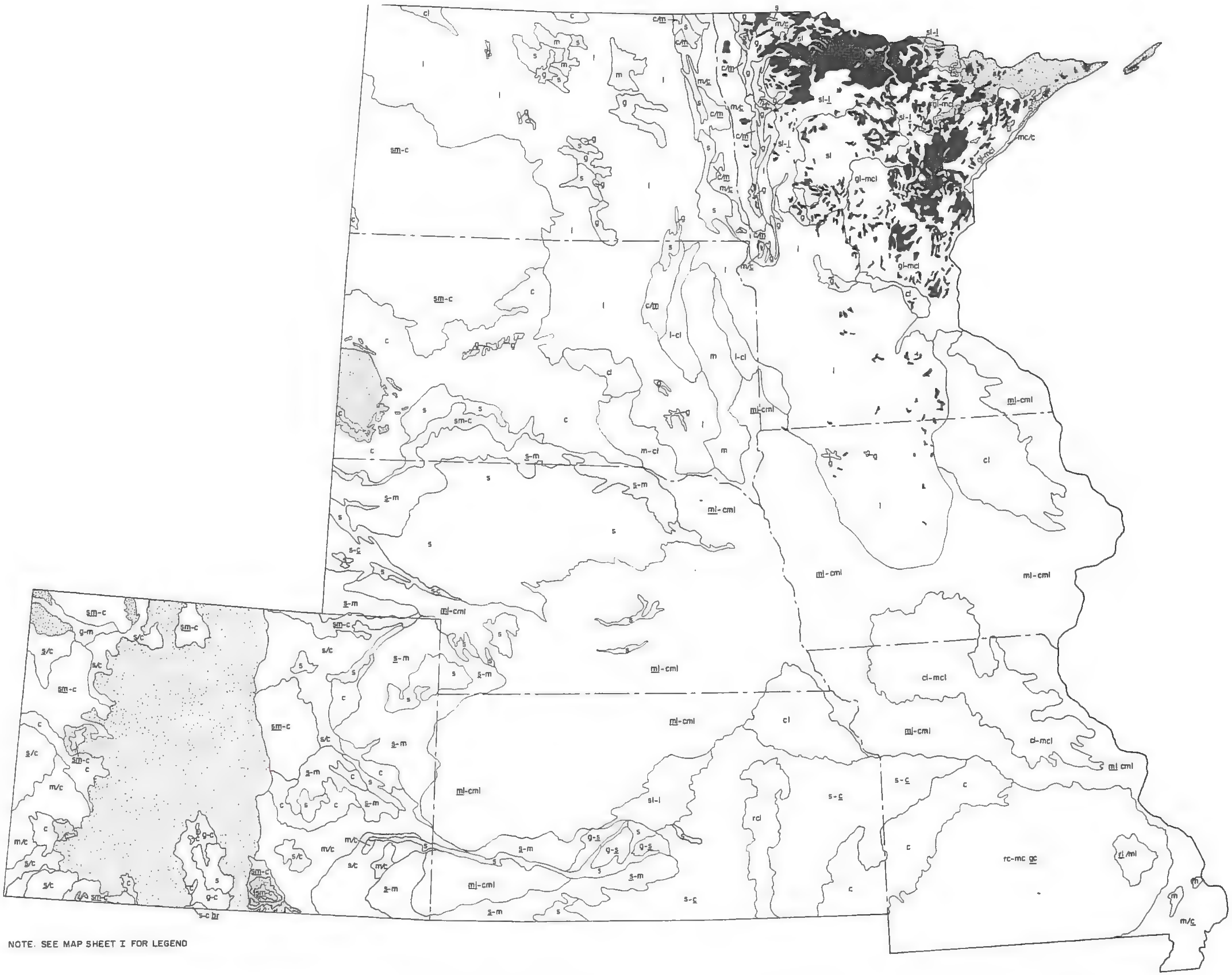


Figure 4. Generalized soil texture map of the continental United States (Map sheet III: south central United States).



NOTE. SEE MAP SHEET I FOR LEGEND

Figure 5. Generalized soil texture map of the continental United States (Map sheet IV: north central United States).

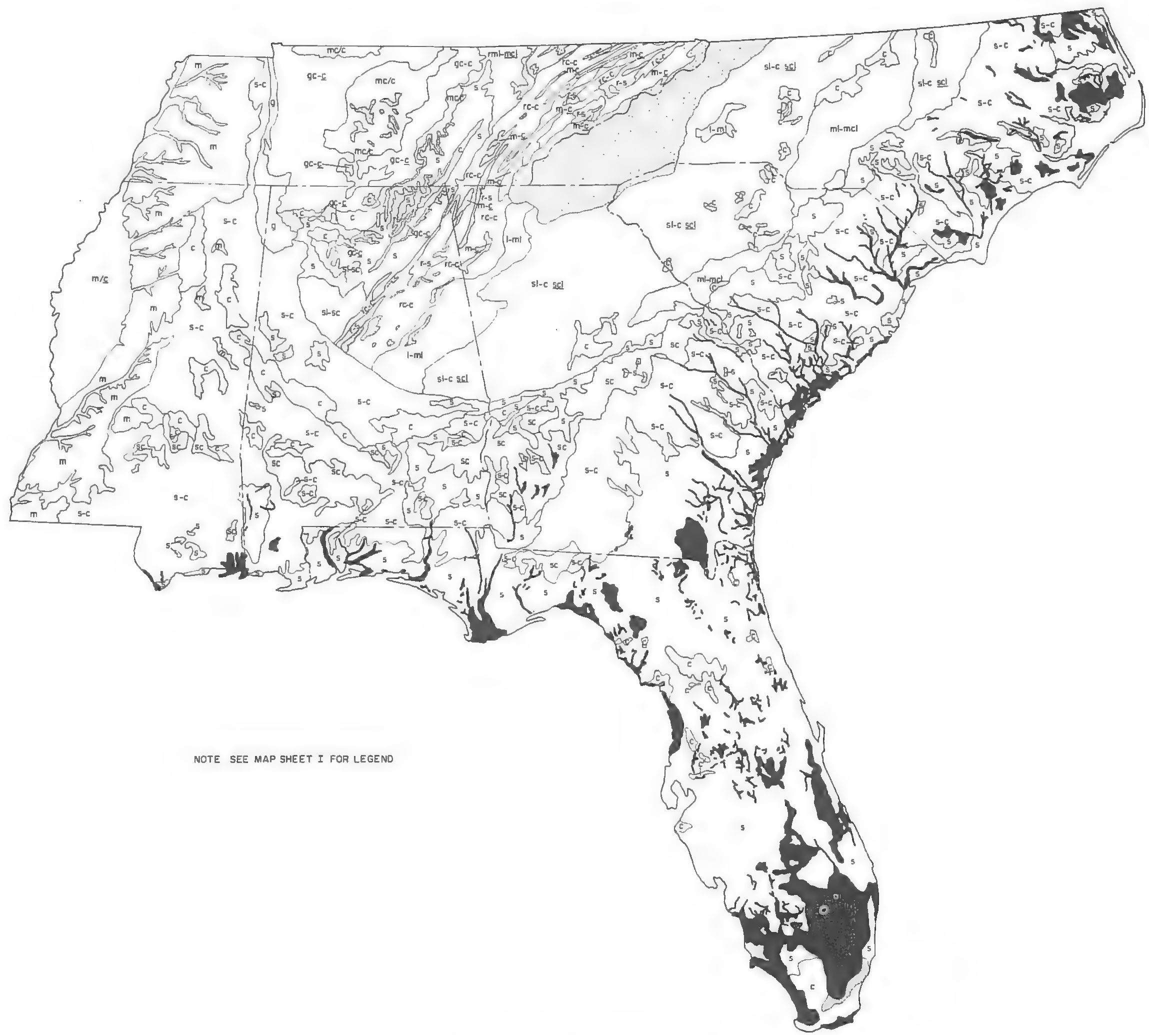


Figure 6. Generalized soil texture map of the continental United States (Map sheet V: southeast United States).

NOTE: SEE MAP SHEET I FOR LEGEND

Figure 7. Generalized soil texture map of the continental United States (Map sheet VI: northeast United States).

TABLE 4

SUMMARY OF ESTIMATED FREQUENCY OF OCCURRENCE OF ORGANIC AREAS WITHIN SECTIONS

Nonexistent		Nonexistent-to-Limited		Limited-to-Medium		Medium-to-Widespread		Very Widespread	
Section Code	Area (sq miles)	Section Code	Area (sq miles)	Section Code	Area (sq miles)	Section Code	Area (sq miles)	Section Code	Area (sq miles)
1c,e	41,830	1a,b,d	60,930	—	—	—	—	—	—
2a,b,c,d	90,670	—	—	—	—	—	—	—	—
3b	4,700	3a,c	37,610	—	—	—	—	—	—
4a,b,c,d	99,000	4e	15,850	—	—	—	—	—	—
5a,b,c,d,e	362,690	—	—	—	—	—	—	—	—
6a,b,c,d,e,f	123,920	—	—	—	—	—	—	—	—
7a,b,c	105,780	—	—	—	—	—	—	—	—
8a,b,c,e	28,200	8d	17,140	—	—	—	—	—	—
9a,b,c	60,450	—	—	—	—	—	—	—	—
10a-m	652,840	—	—	—	—	—	—	—	—
11i	89,580	11b,c,d,f,g	114,730	11a,h	100,590	11e	88,010	—	—
—	—	—	—	12a,b	72,540	—	—	—	—
13a,b,c1,c2,c3	66,490	—	—	—	—	—	—	—	—
14a,b,c,d	51,380	—	—	—	—	—	—	—	—
15a,c,d,e	80,260	15b	22,500	—	—	—	—	—	—
16a,b	45,340	—	—	—	—	—	—	—	—
17a,b	90,670	—	—	—	—	—	—	—	—
—	—	18	6,040	—	—	—	—	—	—
19e,f,g	8,760	19b,c,d	48,940	19a	11,820	—	—	—	—
20e	22,860	20d,f,g	278,420	20a,b	87,180	20c	34,680	—	—
Total	2,025,420		602,160		272,130		122,690		0
Percentage of 48 states	67.0		19.9		9.0		4.1		0

textured units, and gradationally textured units. When soil types occur in combination, and one is known to dominate, it is underlined in the designations. The legend for the soil types is shown in Figure 2 and is self-explanatory.

Organic Deposits

Table 4 summarizes the frequency of occurrence of organic deposits by sections. Figure 8 shows the distribution of the summary table. In addition, the soil texture maps (Figs. 2 to 7) illustrate the actual regional distribution of these deposits.

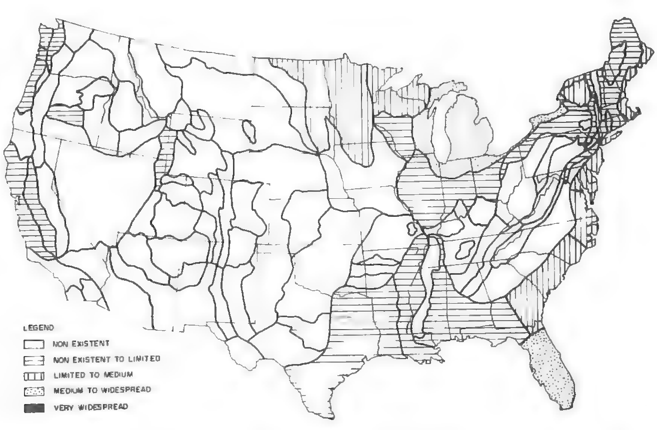


Figure 8. Estimated frequency of occurrence of potential poor subgrade support areas (organic deposits) by physiographic unit.

Clayey Deposits

Table 5 summarizes the estimated frequency of occurrence-severity rating of clayey soils within each section, and Figure 9 shows the geographic distribution of the ratings. The actual regional distribution may be deduced from Figures 2 through 7.

TABLE 5

SUMMARY OF ESTIMATED FREQUENCY OF OCCURRENCE-SEVERITY RATING OF CLAYEY SOIL AREAS WITHIN SECTIONS

Nonexistent		Nonexistent-to-Limited		Limited-to-Medium		Medium-to-Widespread		Very Widespread	
Section Code	Area (sq miles)	Section Code	Area (sq miles)	Section Code	Area (sq miles)	Section Code	Area (sq miles)	Section Code	Area (sq miles)
1c	20,040	1a,b,e	44,180	1d	38,540	–	–	–	–
2a,d	22,940	2b,c	67,730	–	–	–	–	–	–
–	–	3a	14,510	3b	4,700	3c	23,100	–	–
4a,b,c,e	94,290	4d	20,560	–	–	–	–	–	–
–	–	5b,d,e,f	176,630	5a	175,540	5c	10,520	–	–
–	–	6a,b,c,d,e	114,630	6f	9,290	–	–	–	–
7b,c	61,980	7a	43,800	–	–	–	–	–	–
8a,b,c,e	28,200	8d	17,140	–	–	–	–	–	–
9a,b,c	60,450	–	–	–	–	–	–	–	–
–	–	10c,d,e,f,i,j,k	268,310	10a,g,h,1	150,160	10b,m	234,370	–	–
–	–	11g,h	119,060	11d,e	90,370	11a,b,c,f,i	183,480	–	–
12b	8,990	12a	63,550	–	–	–	–	–	–
–	–	13a,c2	12,300	13b,c1,c3	54,190	–	–	–	–
–	–	–	–	14c	16,540	14d	23,580	14a,b	11,260
–	–	15a,e	23,640	15b,c	33,190	15d	45,930	–	–
–	–	–	–	16a,b	45,340	–	–	–	–
17a	19,770	–	–	–	–	17b	70,900	–	–
–	–	–	–	–	–	18	6,040	–	–
19c,d,e,g	20,160	19b,f	37,540	–	–	19a	11,820	–	–
–	–	20a,b,e	110,040	20c,d	126,920	20g	140,480	20f	45,700
Total	336,820		1,133,620		744,780		750,220		56,960
Percentage of 48 states	11.2		37.6		24.5		24.8		1.9

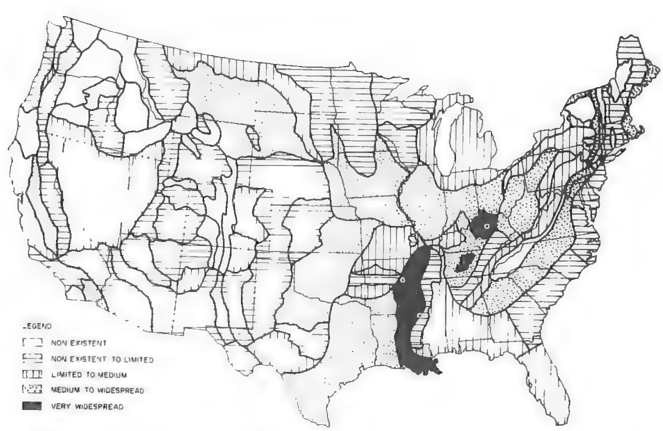


Figure 9. Estimated frequency of occurrence-severity rating of potential poor subgrade support areas (inorganic/clayey deposits) by physiographic unit.

SUMMARY AND CONCLUSIONS

Organic Deposits

The occurrence of organic deposits is relatively limited in the United States. As can be noted from Figure 8, physiographic sections composing an area of almost 87 percent of the conterminous 48 states have, at most, a nonexistent-to-limited rating.

The greatest frequency ratings occur for the Eastern Lakes and Lacustrine Plains Section of the Central and Eastern Lowland Province, and the Florida Section of the Atlantic and Gulf Coastal Plain Province. It is within these two geomorphic conditions-glaciation and coastal plain development—that organic-type terrain becomes a significant factor in highway engineering.

Table 6 summarizes the sections possessing organic deposits, grouped by the major geomorphic modes of occurrence. A salient geomorphic condition for organics is associated with youthfulness on transported deposits. This is due in part to the fact that youthful terrains are often associated with poorly integrated drainage systems.

TABLE 6
SUMMARY OF SECTIONS POSSESSING ORGANIC DEPOSITS GROUPED BY MAJOR GEOMORPHIC MODES

Section Code	Rating[a]	Remarks
I. Glaciated Areas		
Puget Trough (3a)	N-L	
Wasatch (8d)	N-L	found with glacial outwash in Jackson Hole area
Champlain Lowland (11b)	N-L	
Hudson River Valley (11c)	N-L	
Mohawk River Valley (11d)	N-L	
Central Till Plain (11f)	N-L	
St. Lawrence Lowland (11a)	L-M	
Western Lakes and Lacustrine (11h)	L-M	
Eastern Lakes and Lacustrine (11e)	M-W	
Superior Upland (12a)	L-M	
Adirondack (12b)	L-M	
New York Glaciated (15b)	N-L	
Triassic Lowland (18)	N-L	associated with northern glaciated area
New England Upland (19b)	N-L	
Connecticut Lowland (19c)	N-L	
White Mountain (19d)	N-L	
Seaboard Lowland (19a)		
II. Coastal and Embayed Areas		
Oregon Coast Range (1b)	N-L	found within small coastal plain areas of Oregon
California Coast Range (1d)	N-L	associated with section 3c within San Francisco Bay area
California Valley (3c)	N-L	associated with section 1d within San Francisco Bay area
East Gulf Coast (20d)	N-L	occurs primarily in outer coastal plain
West Gulf Coast (20g)	N-L	occurs primarily in outer coastal plain
Embayed (20a)	L-M	
Sea Island (20b)	L-M	
Florida (20c)	M-W	possesses largest swamp area in United States
III. Deltaic Areas		
Mississippi Alluvial Plain (20f)	N-L	associated with Mississippi delta area

[a]N-L = nonexistent-to-limited; L-M = limited-to-medium; M-W = medium-to-widespread.

Clayey Deposits

Table 5 indicates that more than 26 percent of the 48 states have a frequency of occurrence-severity rating of clayey soils more severe than limited-to-medium. Perhaps even more significant is the obvious concentration of clayey areas in the eastern province grouping. (The western province grouping consists of physiographic units having a code prefix of 1 through 9; units composing the eastern grouping are prefixed by numbers 10 through 29.) Approximately 75 percent of the western province grouping area possesses sections having a rating less than or equal to a nonexistent-to-limited severity, whereas only slightly more than 3 percent of the area has sections showing a medium-to-widespread or greater rating. In contrast, in the eastern group almost 40 percent of its area has sections with a medium-to-widespread or greater rating.

There are several probable reasons for this pattern, each perhaps interrelated to the others. It is felt that the major factors are the following:

1. The climatic environment (humid type) prevalent in the East is more conducive to chemical weathering processes that generally are associated with clay development in contrast to physical weathering.
2. The overall topographic features (elevation, relief) of the eastern United States are similarly more favorable for chemical weathering in combination with the climatic regime of the area.
3. The groupings of origin-parent material types in the East are conducive to clay deposition and/or development. Within the glaciated northern portion of the area, the most highly plastic soils are generally associated with water deposition of glacial lacustrine or marine origin. Similarly the clays of the coastal plain are primarily found associated with either the coastal limestones and chalks or widespread fine-grained alluvial deposition. Within these two zones is the consolidated bedrock region that is composed primarily of sedimentary types, in which clayey-type residua are often developed within the climatic and topographic environments peculiar to this region.

Although the physiographic sections used in the analysis are useful in the ratings of poor soil support, they are generally too large and variable to serve the desired purpose. Subsequently, a system of 242 subsections has been developed (19), which should permit more accurate and detailed predictions.

ACKNOWLEDGMENTS

Grateful acknowledgment is made to both NCHRP and Purdue University for allowing this study to be conducted. Special thanks are extended to K. B. Woods, Goss Professor of Engineering at Purdue University, whose outstanding foresight pioneered the advancement of relating engineering experience to physiographic regions.

REFERENCES

1. Cline, M. G. Soils and Soil Associations of New York. Cornell, Extension Bull. 930, May 1961.
2. Corps of Engineers. Forecasting Trafficability of Soils: Airphoto Approach. Tech. Memorandum No. 3-331, Rept. 6, Vol. 2, June 1963.
3. General Soil Map of Florida. Florida Agricultural Experiment Stations, May 1962.
4. Hill, D. E., and Gonick, W. N. The Paxton Soils. Connecticut Experiment Station Bull. 662, Dec. 1963.
5. Hole, D. F., and Beatty, M. T. Soils of Wisconsin: A Generalized Map. Univ. of Wisconsin, Madison, 1957.
6. Jenkins, D. S., Belcher, D. J., Gregg, L. E., and Woods, K. B. The Origin, Distribution and Airphoto Identification of United States Soils. Technical Development Report No. 52, Civil Aeronautics Administration, 1946.
7. Mangum, A. W., et al. Reconnaissance Survey of Southwestern Washington. U. S. D. A. Report, May 1913.
8. Mangum, A. W., et al. Reconnaissance Survey of the Western Part of the Puget Sound Basin, Washington. U. S. D. A. Report, June 1912.

9. Miles, R. D., Scholer, C. F., Shurig, D. G., Woods, K. B., and Yoder, E. J. Regional Factors Influencing Highway Pavement Design and Performance. Unpublished NCHRP Interim Report 1-3(1), Sept. 1965.
10. Prince, A. B., and Raney, W. A. (Review Editors). Some Morphological, Physical, and Chemical Properties of Selected Northeastern United States Soils. Univ. of New Hampshire Agricultural Experiment Station, Misc. Publ. 1, June 1961.
11. Know Ohio's Soil Regions. Ohio Department of Natural Resources, 1962.
12. Soils of the North Central Region of the United States. Univ. of Wisconsin Agricultural Experiment Station, Bull. No. 544, June 1960.
13. Soils and Men. U. S. Dept. of Agriculture, Yearbook of Agriculture, 1938.
14. Certain Properties of Selected Southeastern United States Soils and Mineralogical Procedures for Their Study. Virginia Agricultural Experiment Station, Southern Regional Bull. 61, Jan. 1959.
15. A Key to the Soils of the Coastal Plains of Virginia. Virginia Polytechnic Inst., Agronomy Circular No. 4, Feb. 1963.
16. Soils of the Western United States (Exclusive of Hawaii and Alaska). Washington State Univ., Sept. 1964.
17. Woods, K. B., and Lovell, C. W., Jr. Distribution of Soils in North America Section 9, Highway Engineering Handbook (K. B. Woods, ed.), McGraw-Hill Book Co., New York, 1960.
18. Woods, K. B., Miles, R. D., and Lovell, C. W., Jr. Origin, Formation, and Distribution of Soils in North America. Chapter 1, Foundation Engineering (G. A. Leonards, ed.), McGraw-Hill Book Co., New York, 1962.
19. Witczak, M. W. A Generalized Investigation of Selected Highway Design and Construction Factors by Regional Geomorphic Units Within the Continental United States. PhD thesis, Purdue Univ., Jan. 1970.

PARAMETERS FOR CLASSIFICATION OF FINE-GRAINED LATERITE SOILS OF GHANA

M. D. Gidigasu, Council for Scientific and Industrial Research, Building and Road Research Institute, Kumasi, Ghana

The available literature indicates that engineering properties and field performance of tropically weathered soils are influenced considerably by factors of soil formation, degree of weathering (or laterization), morphological characteristics, and chemical and mineralogical composition as well as environmental conditions. Consequently, temperate-zone highway and airfield soil classification systems based on particle size distribution and plasticity alone are inadequate for these soils. Numerous failures of highway and airfield pavements built with or on laterite soils, in spite of strict adherence to ASTM grading and plasticity specifications, have confirmed the inapplicability of the present temperate-zone classification system. This paper attempts to propose some useful parameters for classification of fine-grained laterite soils of Ghana. The study has revealed that morphology and general characteristics (including chemical and mineral composition) of Ghana soils are influenced considerably by the weathering systems. For a given genetic soil group, the consistency parameters, colloidal activity, compaction characteristics, specific gravity, field moisture equivalent, and linear shrinkage have fairly good correlations with the amount of clay content. The different degrees of weathering (or laterization) of the soil types from the different climatic-vegetational zones have considerable influence on the relation between the clay content and the properties of soils over different parent rocks. The conclusions reached suggest that the residual fine-grained laterite soils of Ghana may be most adequately classified for highway purposes on the basis of such factors as parent materials (geological formations), climatic-vegetational conditions, degree of weathering (or laterization), amount of clay content, and linear shrinkage.

• SOIL classification tests for temperate-climate soils have been standardized for the determination of the particle size distribution and the plasticity parameters. From knowledge of these properties, some general engineering characteristics can be inferred based on information available on other soils of similar classification. In spite of the successful use of this classification system for over 40 years, it is now realized that soil behavior in the field does not depend on particle size distribution and plasticity alone. Certain factors such as genesis, geological history, morphological characteristics, clay mineral type, nature of exchangeable ions, and actual moisture conditions would enhance the technical importance of the existing classification systems.

The use for tropical soils of the temperate-zone classification systems based on particle size distribution and plasticity parameters alone has been very disappointing. In spite of strict adherence to ASTM particle size distribution and plasticity specifications, numerous failures of highway and airfield pavements built on or with laterite

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soils continue to occur in many tropical countries. The adoption of particle size distribution and plasticity alone for classification of laterite soils has proved unsuccessful for three reasons:

1. These classification tests do not yield reproducible results because they are influenced considerably by methods of preparation and handling of the material (10, 42, 48, 56).
2. Because the material is a decomposition material, it may contain materials with different degrees of weathering. Thus, these tests could not yield adequate indications of the engineering properties without some definition of the degree of weathering (or laterization) (4, 19, 44).
3. The engineering properties and field performance of laterite soils are influenced considerably by chemical and mineral content, genesis, morphology, and the environment (7, 17, 24, 51, 57).

The problem of classifying laterite soils is to evolve a simple system that takes account of the fact that the material is a decomposition (or laterizing) material with different degrees of weathering (or laterization) and different chemical and mineralogical contents formed in different environments. The application of the system should involve the use of simple tests that yield reproducible results. One such simple test was found useful for identifying fine-grained laterite soils in linear shrinkage (6, 25, 27, 55).

In this paper an attempt is made to propose some useful parameters for classification of fine-grained laterite soils of Ghana. It is hoped that this study will contribute toward a more rational and realistic approach to the identification and engineering classification of tropically weathered soils.

REVIEW OF LATERITE SOIL CLASSIFICATION SYSTEMS

The greatest problem encountered in the study of laterite soils is the difficulty of evolving a definition or classification system acceptable to chemists and geologists as well as soil scientists. Although there are numerous factors that influence the formation of laterite soils in tropical and subtropical climatic conditions, three basic criteria based on common characteristics have been identified by the soil scientists, chemists, and geologists. These criteria are the physical property of in situ hardening or hardening on exposure to air (14), chemical composition (26, 33, 36, 41), and the morphological characteristics of the laterite soil profiles (45).

The difficulties of defining and classifying laterite soils have been carried over to engineering studies. Different methods of classifying laterite soils have been proposed. The first attempt to group laterite soils for engineering purposes was based on the pedogenic characteristics of the soils (16). Little (38) proposed a method of classification based on morphological characteristics and on an estimate of the degree of weathering on the parent rocks, following the work of Lumb (39). This is of limited application, however, to laterite soils because the formation of these soils is complicated by the enrichment of iron or aluminum or both.

Classification systems based on climatic vegetational conditions, morphology, topography, and drainage conditions were also suggested (9, 15, 17, 46). Laboratory studies backed by field experience in many African countries (28) have revealed that none of these methods is completely adequate for general application. The fact that some success was noted with some of these methods in some countries only reveals the importance of these factors to the engineering properties and field performance of laterite soils.

FORMATION OF SOILS IN GHANA

The aim of the study reported here was to determine the most important parameters by which a highway soil classification system could be established for the laterite fine-grained soils of Ghana. To accomplish this, it was necessary to investigate the formation and distribution of these soils as determined by such factors as climatic-vegetational conditions, parent rocks, topography, and drainage conditions.

Geology and Distribution of Rock Types

Figure 1 is a simplified geological map of Ghana. It shows that the most important rocks are gneisses in the coastal savanna zone; granites and phyllites in the semideciduous forest zone; and granites, sandstones, and shales in the deciduous woodland savanna zone. The geomorphology and physical features depend mainly on the geology; most of the hills and ranges consist of hard resistant rocks such as quartzites, whereas the valleys and lower grounds are carved out of softer rocks such as shales, sandstones, phyllites, and schists.

Climate and Vegetation

The rainfall, temperature, and relative humidity are generally very high, as is typical of a tropical climate. The vegetation depends mainly on the intensity and distribution of rainfall.

Based on rainfall intensity and vegetation type, Ghana was roughly divided into three climatic-vegetational zones. The coastal savanna zone receives average annual rainfall of less than 1,000 mm, whereas the corresponding values for deciduous woodland savanna and the semideciduous forest zones are 1,000 to 1,250 mm and above 1,250 mm respectively (Fig. 2).

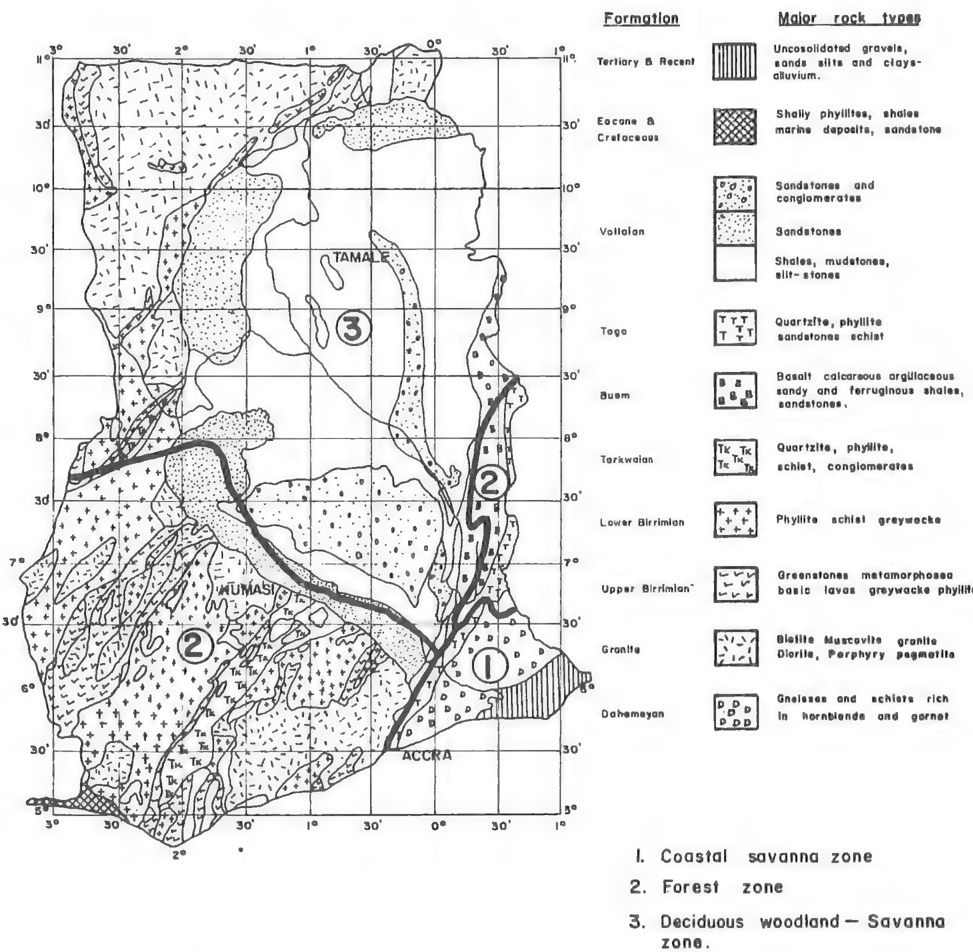
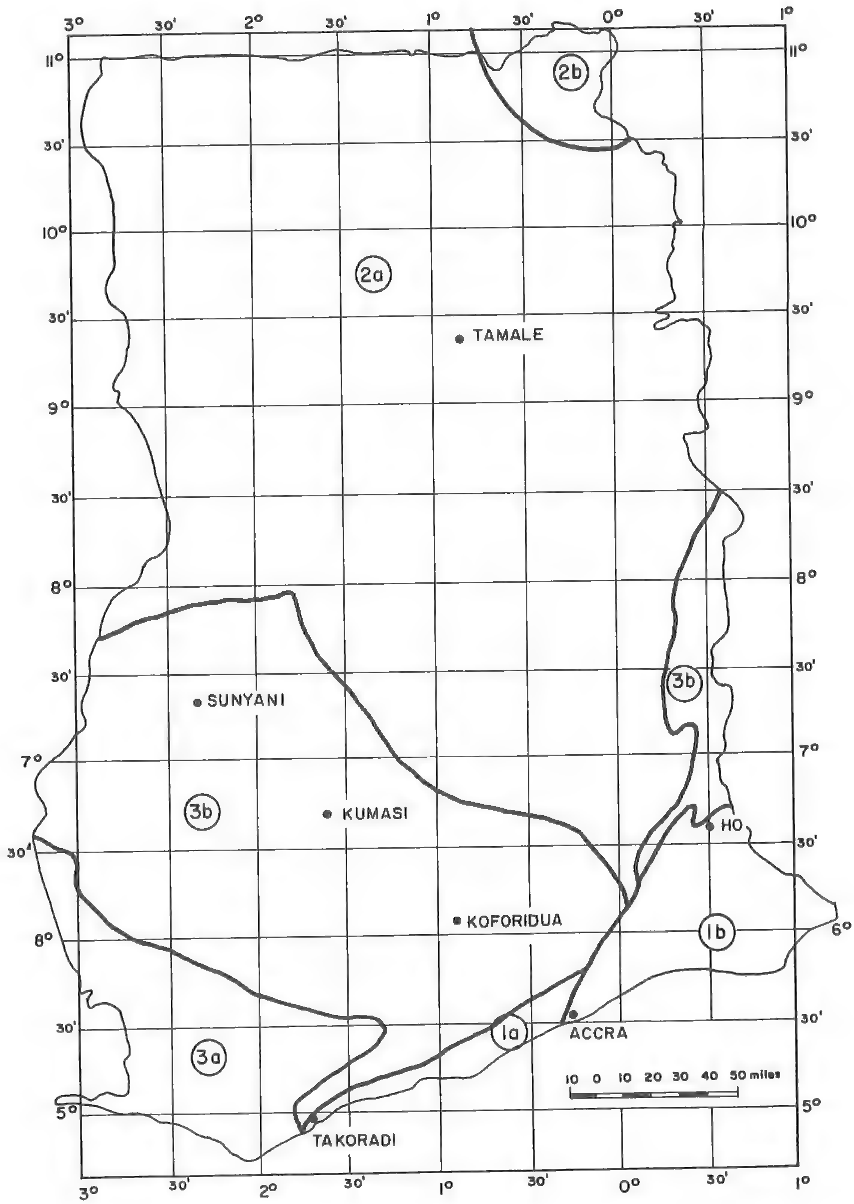


Figure 1. A simplified geological map of Ghana [after Bates (11)].



1a – Coastal thicket
1b – Coastal savanna } Cs = Coastal savanna zone

2a – Guinea savanna
2b – Sudan savanna } Ws = Deciduous woodland savanna zone

3a – Rainforest
3b – Semideciduous forest } F = Forest zone

Figure 2. A simplified climatic-vegetational map of Ghana.

Tropical Weathering and Laterization

Weathering is started by the physical breakdown of parent rocks resulting from differential expansion and contraction of the rock-forming minerals due to temperature variations.

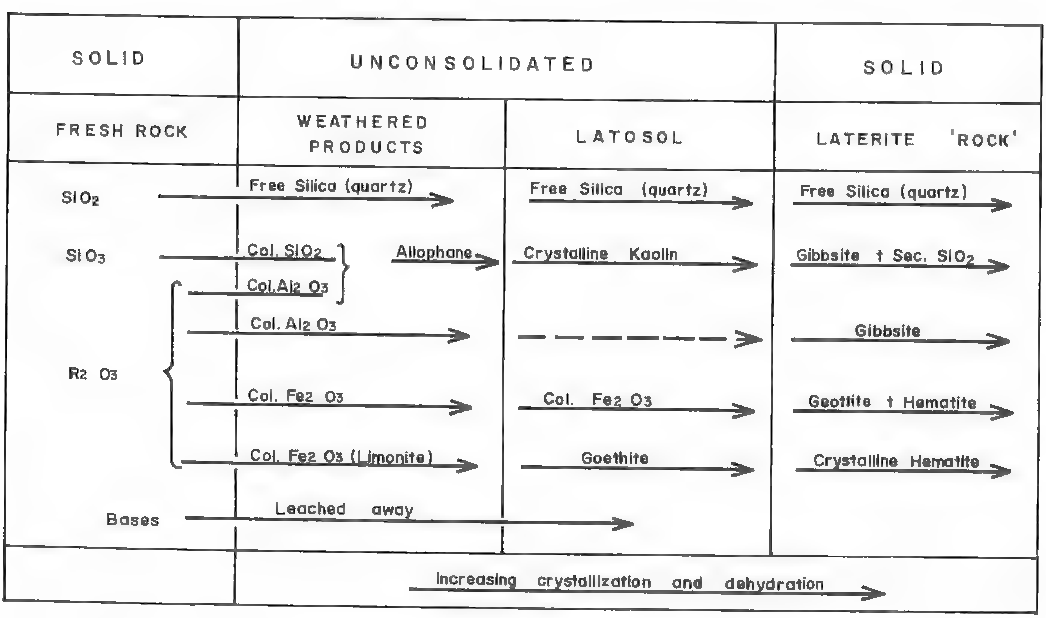
The rock particles, however, still retain their original chemical and mineralogical compositions. Chemical weathering involves the chemical breakdown of unstable primary minerals and the formation of new minerals. Water containing biochemical products of vegetable decay and other corrosive compounds attacks the parent rock pieces. The weathering process is selective; less stable minerals such as biotite are attacked first, whereas quartz is more resistant. The weathering process advances until only very stable primary minerals are left. This summarizes a very complex weathering process that is affected by the factors of soil formation, parent rocks, climatic-vegetational conditions, local topography, and drainage conditions.

The process of laterization that follows involves the leaching in solution of the weathered materials, mainly combined silica and bases, which leaves the residue relatively rich in hydroxides and oxides of aluminum, iron, and titanium (laterite constituents). Sometimes soluble laterite constituents migrate themselves; good internal drainage is a very important factor here.

The presence of laterite constituents in the parent rocks or in adjacent uplands is thus one of the most important prerequisites for the laterization process to take place. In an earlier study (27) it was shown that most Ghanaian rocks contain fairly large amounts of laterite constituents, which accounts for the fact that laterite profiles are found on most of these parent rocks.

The mineralogical stages of laterization of soil over granite under Ghanaian conditions were investigated (30), and the results are shown in Figure 3. It is seen that, as laterization advances, the degree of crystallization and dehydration of the laterite constituents increases.

The literature has emphasized the chemical, mineralogical, and physical (hardening) aspects of laterization; however, the engineering significance of the physicochemical



Col. = Colloidal. $R_2O_3 = Al_2O_3 + Fe_2O_3$, Bases = CaO, MgO, Na_2O, K_2O, Sec = Secondary.

Figure 3. Chemical and mineralogical stages of laterization under Ghanaian conditions [after Hamilton (30)].

aspect involving the coating of the soil particles by free alumina or iron oxide gels has been ignored. It is assumed in this study that the coating of the soil particles is one of the most important factors that underlines the physicochemical and physical differences between laterites and temperate-zone soils. The author maintains that the coating considerably influences the surface activity of the clay minerals and hence of soils containing these minerals.

The cementation of the laterite soil is due to the presence of free iron. The free iron oxide is generally found in three different forms: hematite, limonite, and geotite. The increasing degree of laterization results in an increase of the thickness of free iron oxide (hematite) coating of the soil particles (42). The soil particles later coagulate into larger clusters with subsequent reduction in specific surface. It is the reduction in specific surface that is of interest in soil mechanics.

DISTRIBUTION AND GENERAL CHARACTERISTICS OF SOILS IN GHANA

The field identification studies carried out by the author combined with available pedological, geological, and engineering information revealed that the pedological soil map proposed by Branner and Wills (13) gives the best picture of the distribution of subsoil types in Ghana. Because this soil classification is based on the consideration of soil-forming factors, the soil map forms a sound basis for the preliminary classification of soils. The pedological soil map was therefore simplified by considering modes of formation and morphological characteristics as well as texture and color of soils (Fig. 4).

Variation of Soil Color With Local Topography

The color of soil (below the humus topsoil) changes from red or reddish brown on the summits and upper slopes through orange-brown on the middle slopes to yellow-brown on the lower slopes. These color variations reflect changes in the degree of hydration of iron or alumina present in the profiles, which depend on changes in internal drainage conditions in different parts of the local topography. In poorly drained areas the soil color varies from yellow, through grey-black to white. Where soil profiles are subjected to alternate reduction and oxidation (due to waterlogging and drying out respectively) mottles are formed (patches of yellow, orange, or red in an otherwise grey soil profile). A rough assessment of the degree of laterization in terms of color of the soil based on the available literature and field studies is suggested as follows:

1. Coastal zone tropical clays over gneiss are of low degrees of leaching and laterization;
2. Coastal zone reddish residual soils over gneiss are of medium degrees of leaching and laterization; and
3. Forest zone residual soils over granites and phyllites are highly leached and laterized.

Chemical Characteristics

Regarding the chemical characteristics of Ghana soil groups, Figure 5 shows that there is fairly good negative correlation between the total silica and the sesquioxides for the nonresidual soils, tropical clays, and residual soils; however, the relation breaks down for the laterite rocks. The total sesquioxide content of the nonresidual soils and tropical clays ranges between 0 and 20 percent. For residual laterite soils and the laterite rocks, corresponding values are between 20 and 50 percent and more than 50 percent respectively. The large silica content in the nonresidual and tropical clay soils may be due to the concentration of dissolved silica in these profiles and to the very finely divided quartz remaining in the residual wash that has been carried to lower flat grounds.

Mineral Characteristics

Results of petrographic and mineralogical studies on different soil types of the various pedological groups (29, 30, 47, 50) and at the Ghana Geological Survey Department

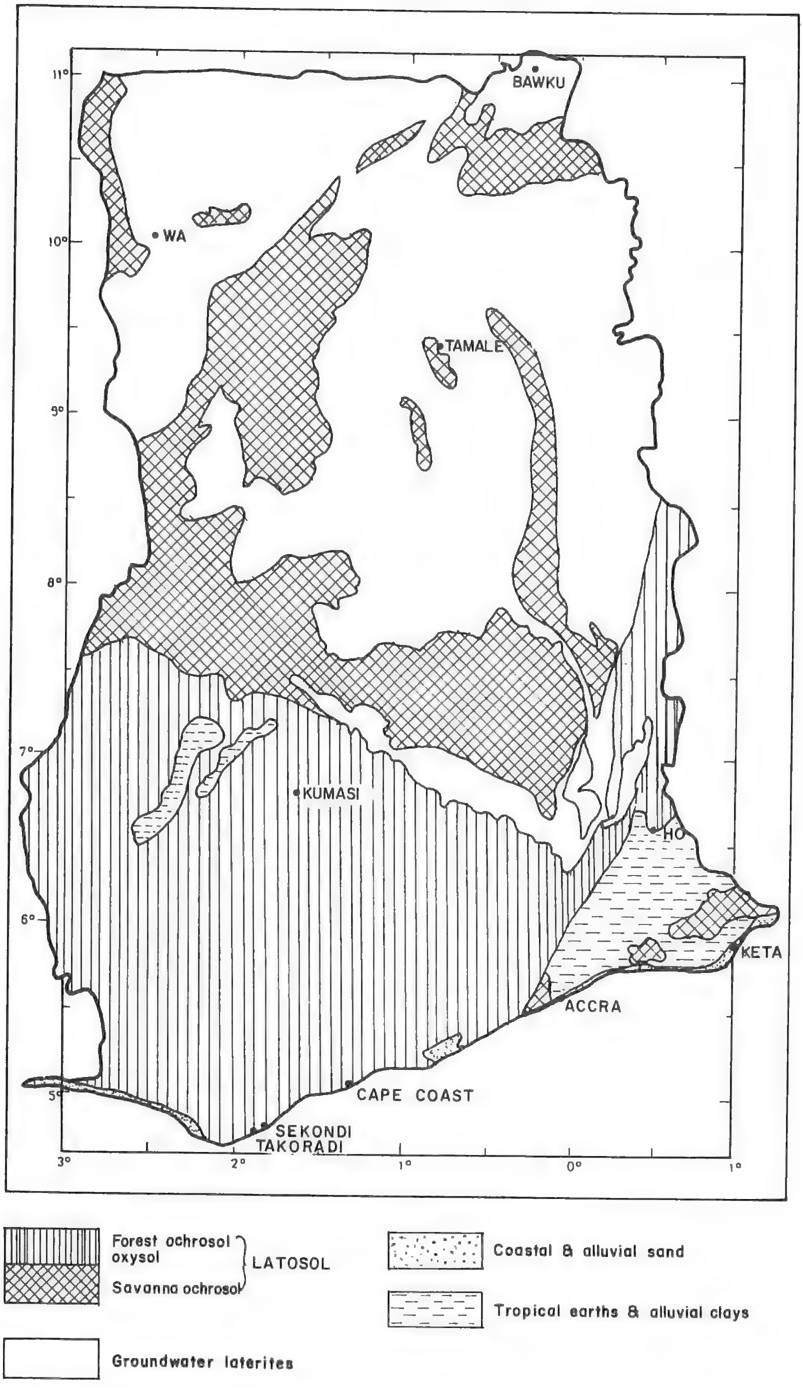


Figure 4. A simplified pedological map of Ghana [after Branner and Wills (13)].

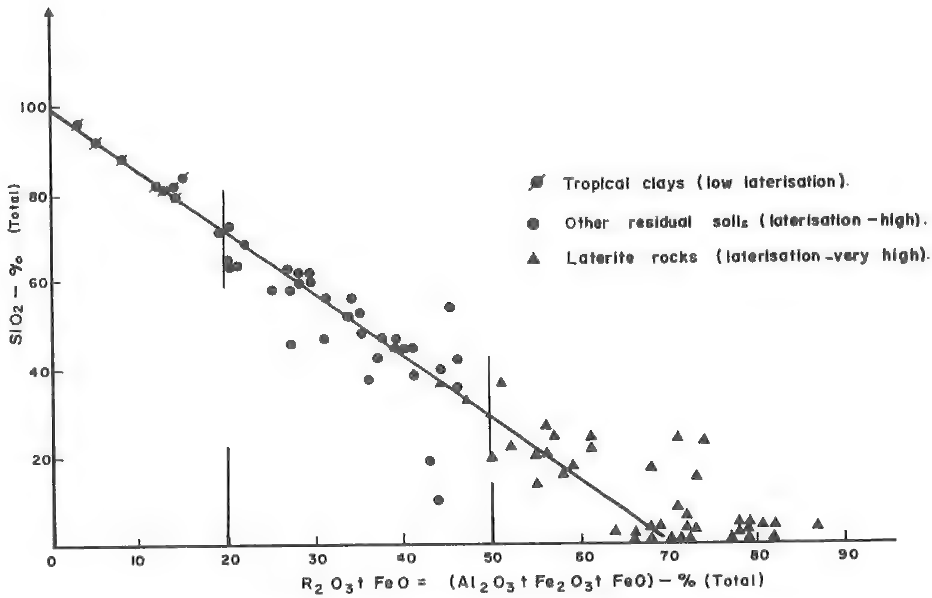


Figure 5. Generalized chemical characteristics of soils in Ghana.

(for the Building and Road Research Institute) have revealed that, owing to the intensity of the chemical weathering processes under Ghanaian conditions, all the unstable primary minerals are either partially or completely broken down and/or transformed into secondary minerals. The most frequently encountered unweathered common primary minerals in the laterite profiles are quartz, garnet, staurolite, illite (muscovite), feldspar (microcline), hornblende, and rutile; these minerals are stable under laterite conditions. The particular mineral available depends on the parent rock type. The chief clay mineral observed was kaolinite, with mica (sericite and illite) in the latosols. Montmorillonite and sometimes vermiculite are also present in the poorly drained tropical clays. The secondary minerals resulting mainly from the laterization processes are gibbsite, geotite, limonite, and hematite. Sometimes magnetite was noticed, but neither manganese nor titanium minerals were observed in significant amounts.

BASIC GROUPING OF LATERITE SOILS

On the basis of field and laboratory experience with laterite soils in Ghana and elsewhere, the laterite soils of Ghana may be texturally classified into three main groups, as shown in Figure 6. All textural groups may be either residual or nonresidual formations. The three groups are as follows:

1. Laterite rocks and boulders—These are concretionary laterite rocks and rock pieces that are extensively used in some African countries as concrete and road surfacing aggregates. Because natural aggregates are fairly abundant in Ghana, however, very little interest has been shown in these materials, and they are therefore not covered in this study.
2. Laterite and laterite-quartz gravels—These gravels form the most important naturally occurring gravels and are extensively used as base and subbase materials in Ghana. These materials have therefore been extensively studied. Some proposals on the classification of these gravels have already been published (19), and further studies are in progress; these gravels are therefore not considered in this paper.
3. Laterite fine-grained soils—These soils have been given very little attention in the past, but recent highway subgrade and shallow foundation failures have em-

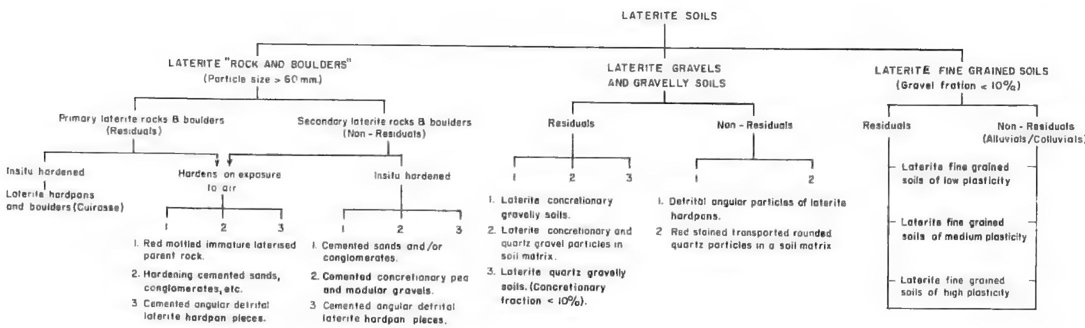


Figure 6. Pedogenic-textural groups of laterite soils in Ghana.

phasized the importance of studying and understanding them. The study is restricted to this soil group.

Because highway construction and housing schemes are being concentrated at the moment in the southern part of the country (coastal savanna and the forest zones), the soils from this region have been selected for study. Only residual fine-grained soils were studied in this program. In order to isolate the effect of depth, the samples were taken from the first 6 ft below the ground surface.

SELECTION OF SOILS FOR STUDY

In selecting the fine-grained soils for the studies, consideration was given mainly to the occurrence of the soil types and their importance as road-making materials. The latosols developed over granites and phyllites in the forest zone form the most widely distributed road-making materials in Ghana. Although they are not widely distributed, the coastal savanna residual gneissic soils and tropical clays form a group of slightly expansive to highly expansive soils.

In all, 97 soil types were examined in this study. They are distributed as follows: 31 soils over gneisses in the coastal savanna zone, 29 soils over granites, and 37 soils over phyllites in the forest zone. The locations of the soils are shown on the pedological soil map in Figure 7.

LABORATORY TESTS

The following four genetic soil groups were subjected to detailed laboratory tests:

1. Coastal zone tropical residual black and brown clays over gneiss (Cs-Gn-Tc),
2. Coastal zone reddish residual soils over gneiss (Cs-Gn-R),
3. Forest zone reddish residual soils over granite (F-Gt-R), and
4. Forest zone reddish residual soils over phyllite (F-Ph-R).

The physical properties determined were particle size distribution, consistency parameters, specific gravity, optimum moisture content and maximum dry density (standard Proctor), field moisture equivalent, and linear shrinkage.

The particle size distribution, consistency, specific gravity, and compaction tests were carried out according to the specifications described in British Standard No. 1377 (1961). The field moisture equivalent was determined according to the procedure proposed by Hogentogler (31).

Linear shrinkage was determined on soil fractions that were passed through a B.S. No. 36 sieve, oven-dried, and mixed thoroughly with distilled water to a moisture content approximately equivalent to the liquid limit. The soil was packed into a waxed metal trough 0.5 by 0.5 by 6 in. and then oven-dried to constant weight. The decrease in length of the paste after oven drying is expressed as a percentage of the 6-in. dimension.

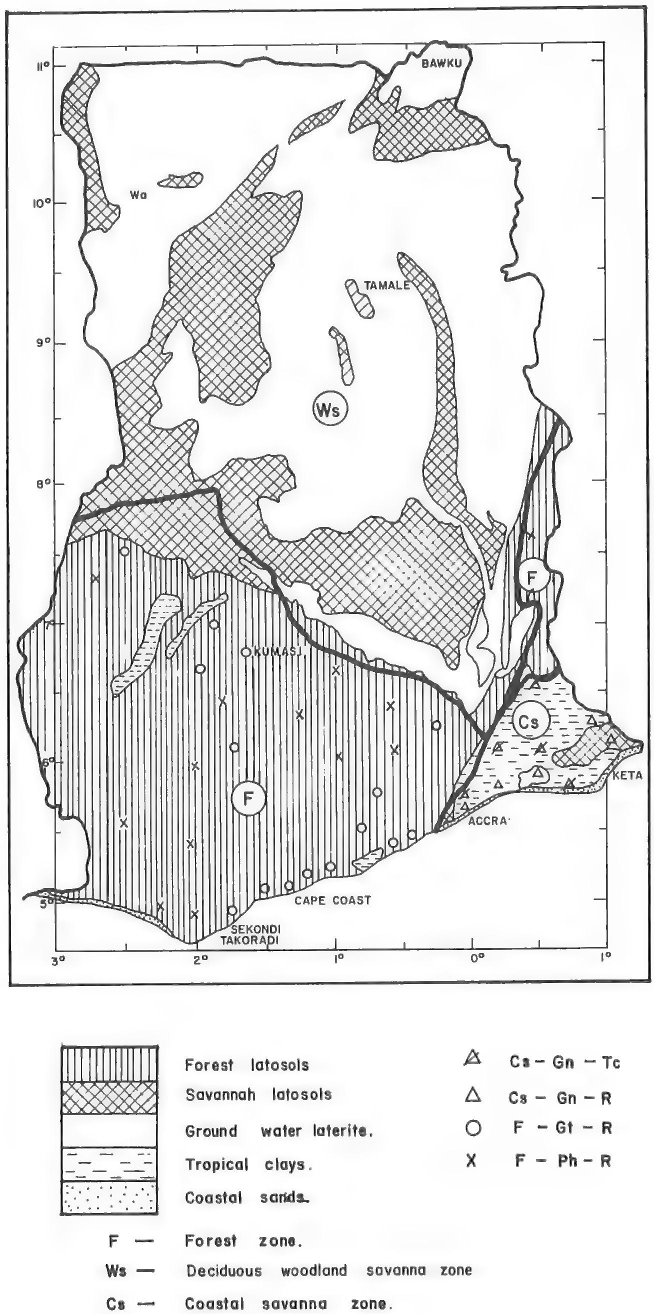


Figure 7. Locations of the soils studied.

Discussion of Test Results

The physical test results were interpreted in the light of the following factors: (a) climatic-vegetational zones, (b) parent rocks, (c) degree of leaching and laterization, (d) clay content, and (e) linear shrinkage.

Texture of the Soils—The textural classification of the soil groups over the different parent rocks is shown according to a triangular chart in Figure 8. Tropical clays range from sandy loams to heavy clays. The other residual soils over gneiss are mainly sandy clay loams, sandy loams, and loams. Granite soils range texturally from sandy loams and clay loams to lean clays.

Phyllite soils on the other hand are richer in silt and clay fractions. The most important soil types developed over phyllite are clays and clay loams. Although the texture of these soils is influenced to some extent by the degree of weathering, the petrographic characteristics of parent rocks seem to account more for the textural differences in the soils over the different parent rocks.

Consistency Characteristics—Figure 9 shows the relation between liquid limit and clay content for the different soil groups. It is seen that there is a fairly good correlation between the two properties. The influence of the clay content on the liquid limit is more pronounced for the tropical clays (soils of low degree of laterization) than for other fairly laterized soils.

Figure 10 shows the relation between plastic limit and the clay content for the different soil groups. Except in the case of phyllite soils, the clay content does not seem to influence the plastic limit in the soils studied.

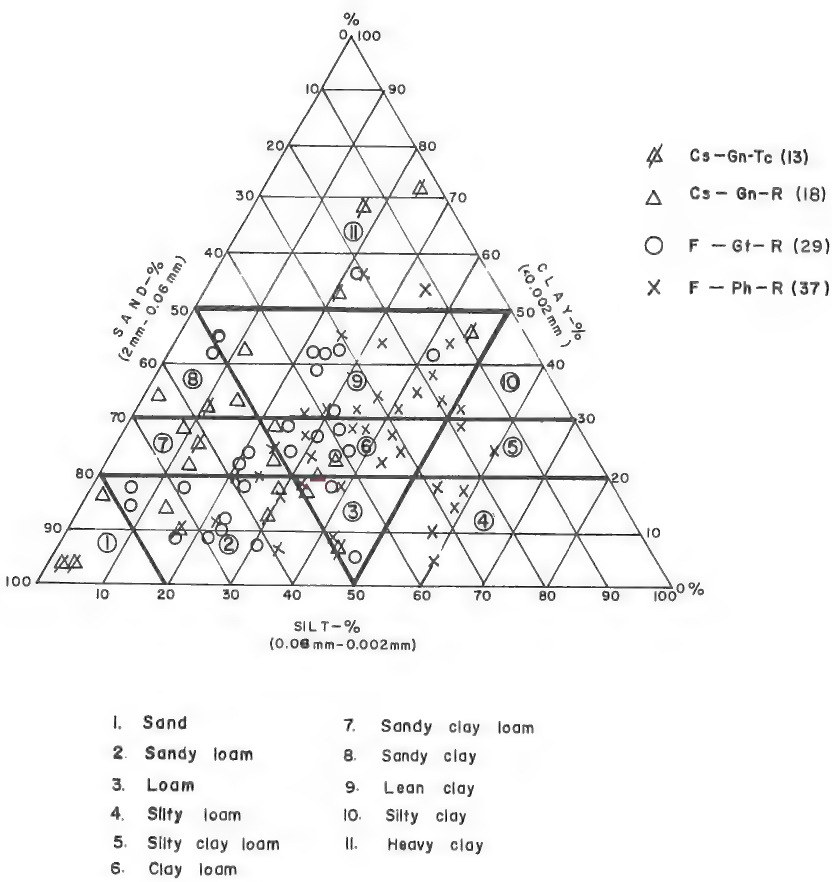


Figure 8. Textural classification of the soil types studied.

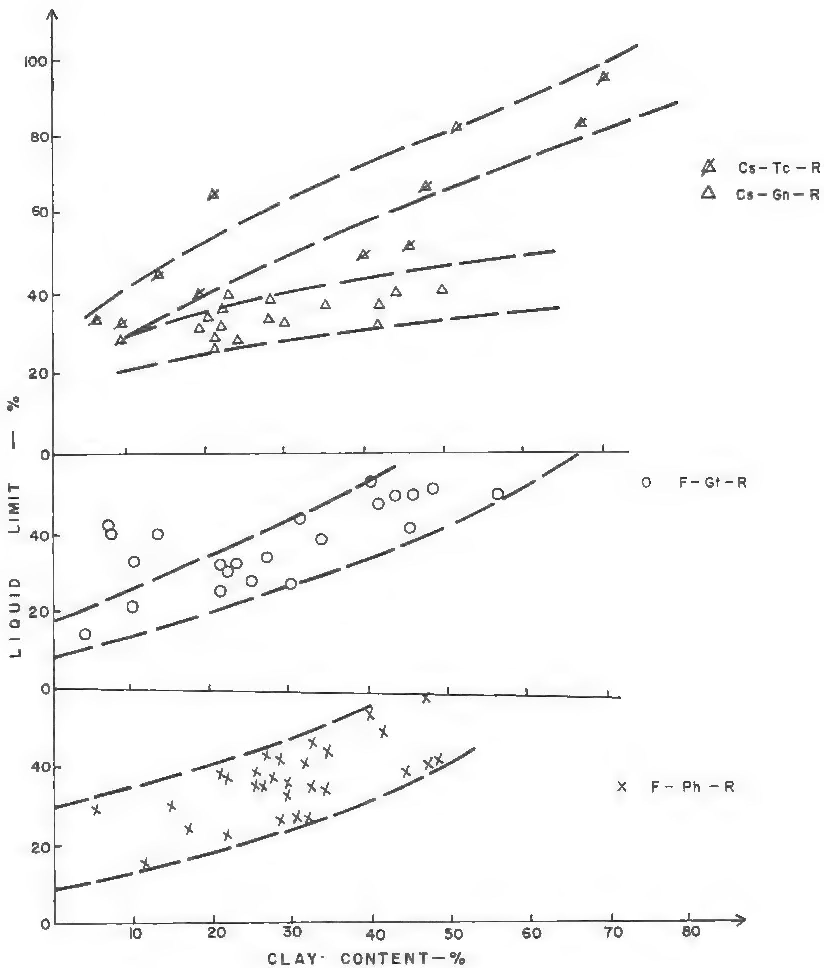


Figure 9. Relation between clay content and liquid limit.

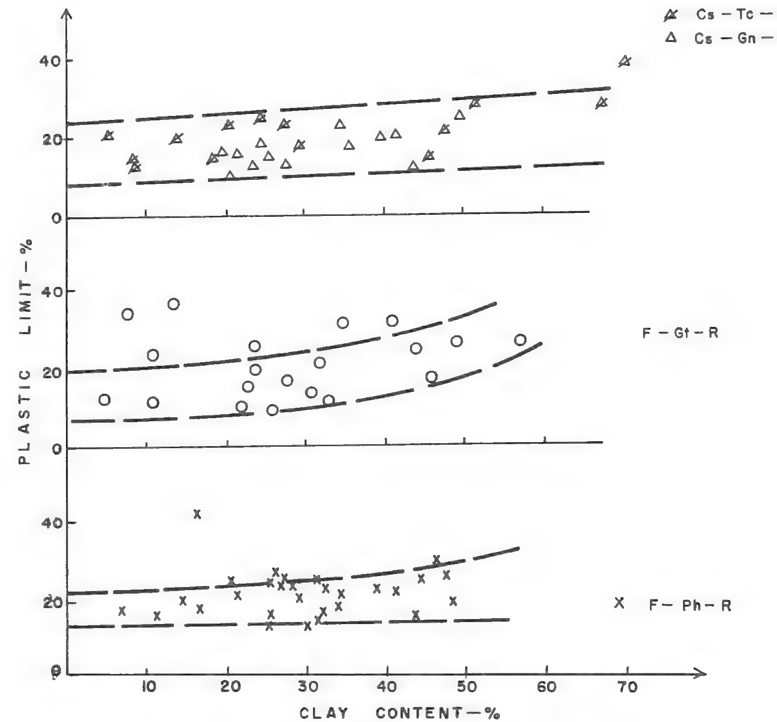


Figure 10. Relation between clay content and plastic limit.

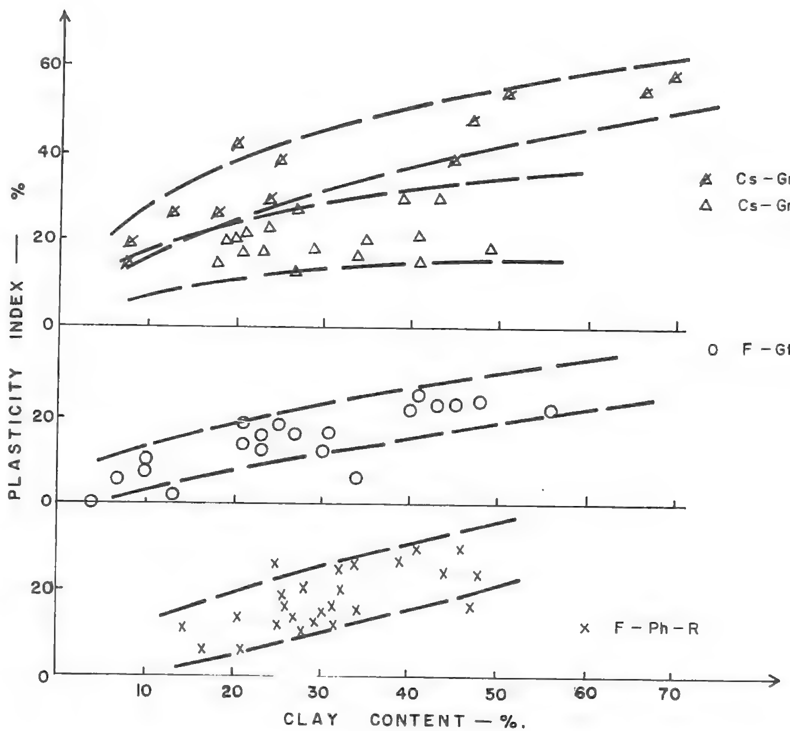


Figure 11. Relation between clay content and plasticity index.

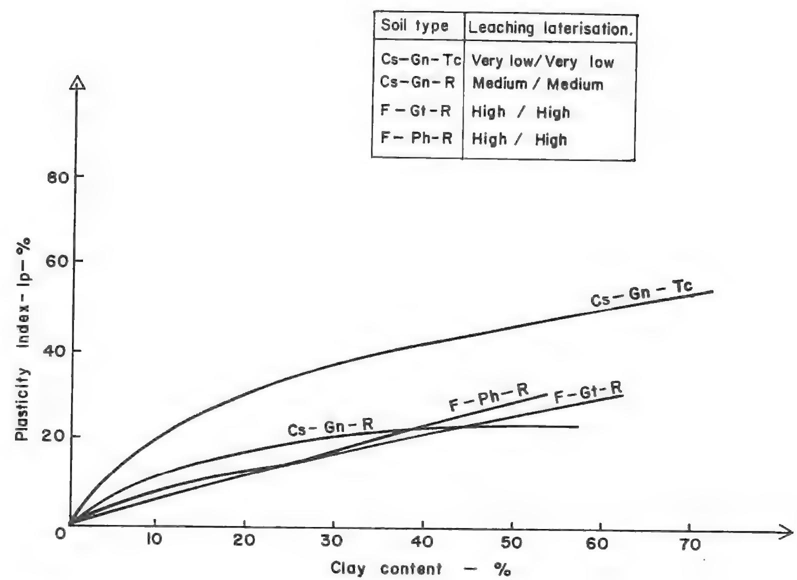


Figure 12. Average curves of the relation between the clay content and the plasticity index.

The plasticity index gives the percentage of water in relation to the dry soil that is required to change the soil from the semisolid state to the flow state. This moisture property is very important because it reflects the amount and type of clay, mineralogical composition, exchangeable ions, and base exchange capacity in the soil. In a sense it is a parameter by which the potentiality of a soil to absorb water is assessed. Figure 11 shows the relation between plasticity index and clay content for the soil groups. There is a fairly good correlation between the two properties for all the soil groups. From the average curves in Figure 12 it is seen that the increase in the plasticity index with clay content is more pronounced for the tropical clays, compared to the other laterite residual soils. This illustrates the fact that soils of a low degree of laterization (the tropical clays) are influenced more in terms of plasticity by the amount of clay content.

Figure 13 shows the plasticity characteristics of the soil groups on the Casagrande chart. It is seen that residual soils that generally fall above the A-line have liquid limit values mainly below 60 percent. Higher liquid limit values were recorded for tropical clays. Micaceous granite residual soils that have been found to fall below the A-line are generally of medium plasticity.

Figure 14 shows the relation between clay content and colloidal activity for the soil groups. For all except the tropical clays, colloidal activity ranges between 0.4 and 1.0. For tropical clays, the value is generally more than 1.0. It is seen that colloidal activity is higher for the less laterized tropical clays compared to the other soil groups. The differences in colloidal activity values can also be explained in terms of soil genesis. Tropical clays, by virtue of their mode of formation, contain some montmorillonite (50), which is responsible for the higher colloidal activity. The forest-zone highly laterized soils with kaolinite type of clay mineral (29) have activity ranging between 0.4 and 1.0. The relation between clay content and field moisture equivalent is shown in Figure 15.

Compaction Characteristics—A fairly good correlation was found between clay content and optimum moisture content (standard Proctor) and maximum dry density. The relation between clay content and optimum moisture content is shown in Figure 16. No

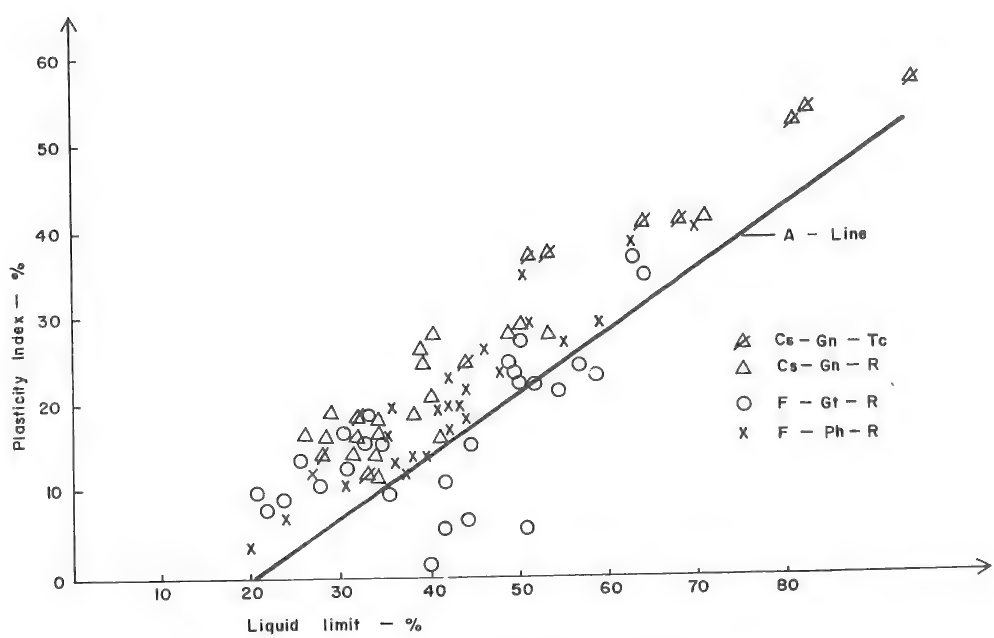


Figure 13. Plasticity characteristics of the soils studied.

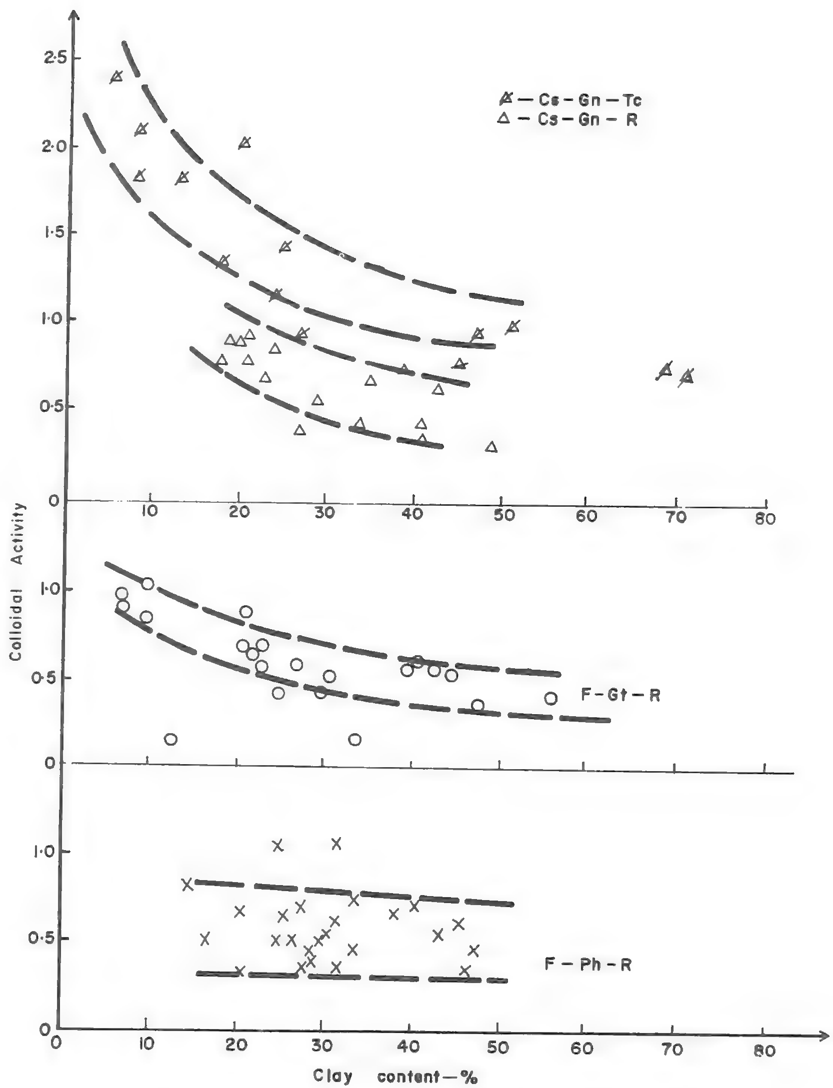


Figure 14. Relation between clay content and colloidal activity.

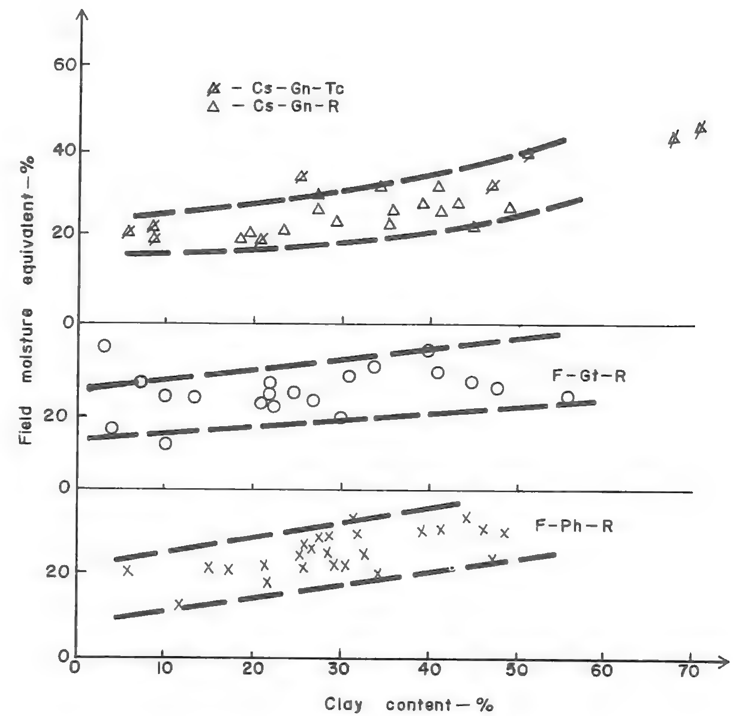


Figure 15. Relation between clay content and field moisture equivalent.

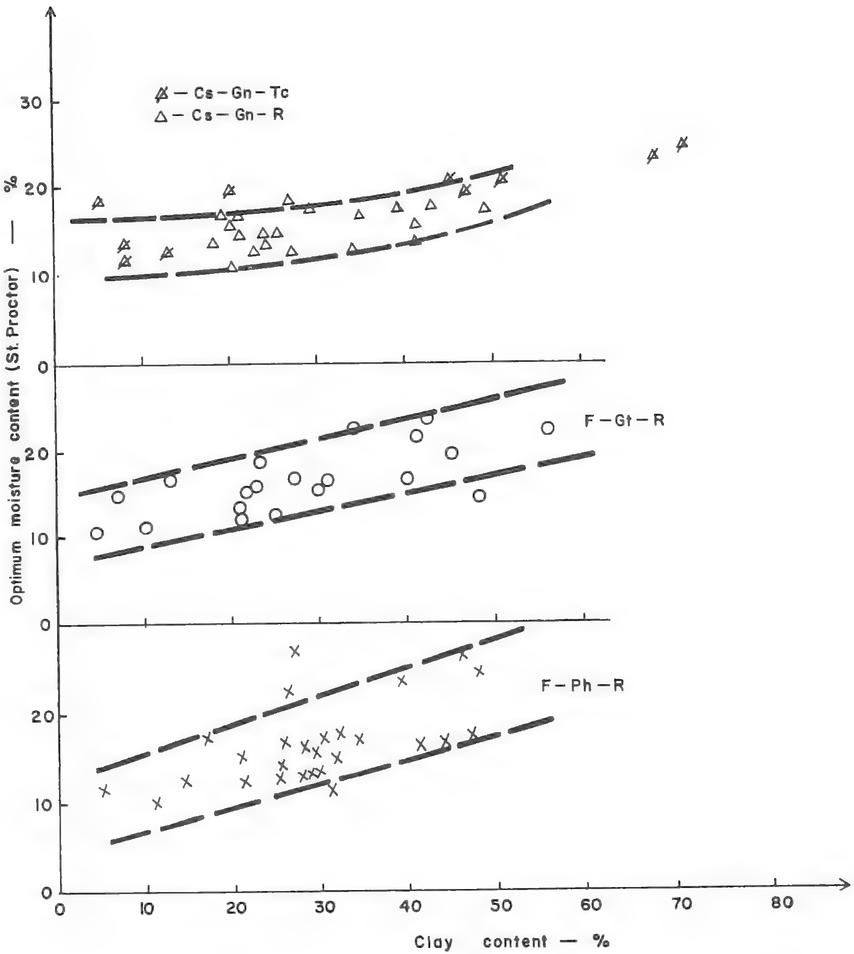


Figure 16. Relation between clay content and optimum moisture content (standard Proctor).

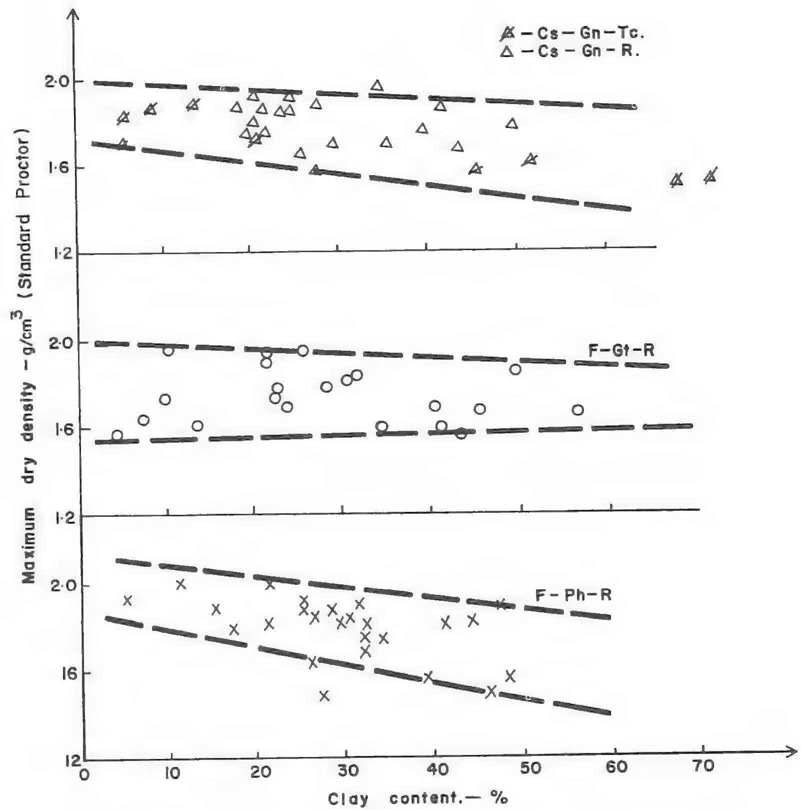


Figure 17. Relation between clay content and maximum dry density (standard Proctor).

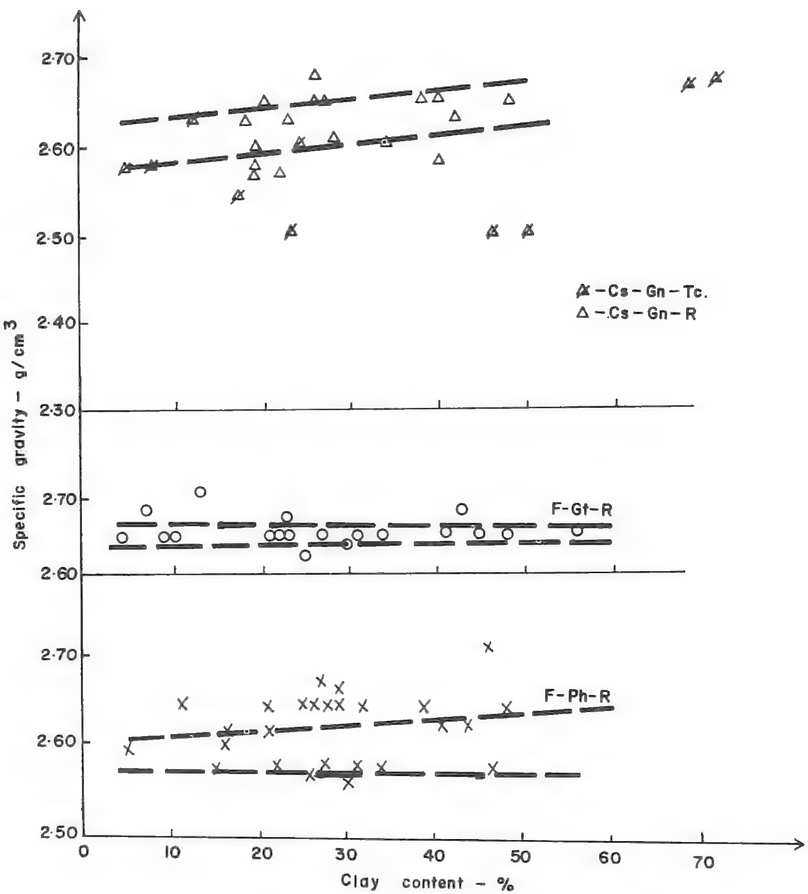


Figure 18. Relation between clay content and specific gravity (for fraction smaller than 2 mm).

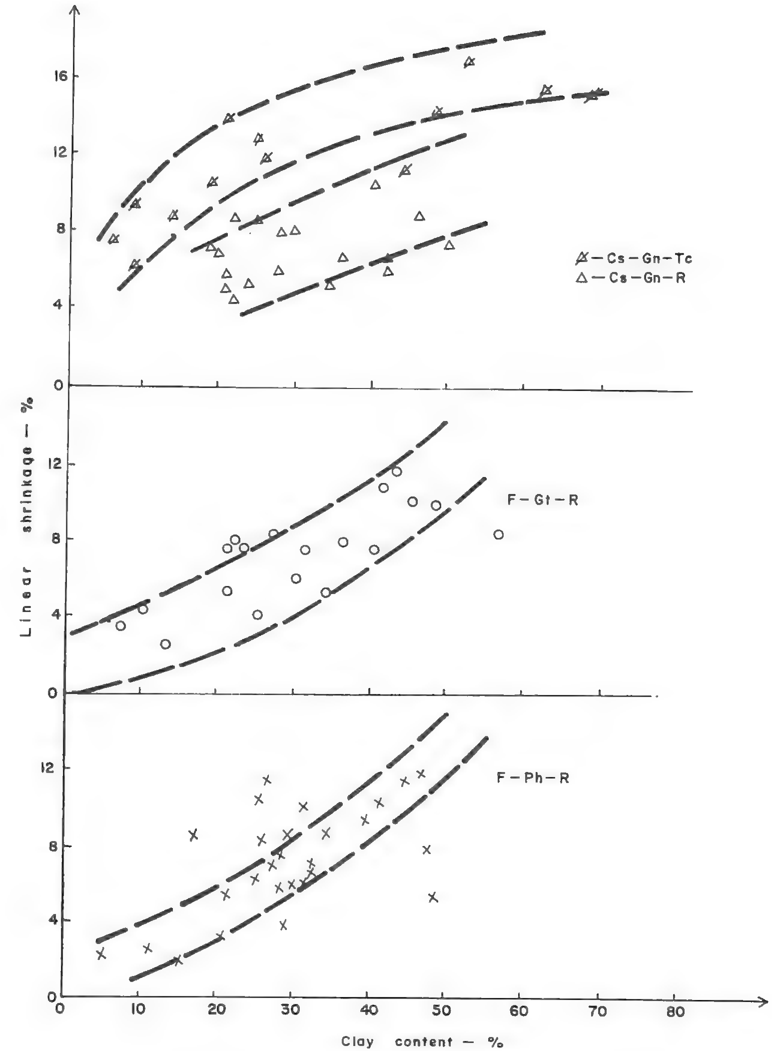


Figure 19. Relation between clay content and linear shrinkage.

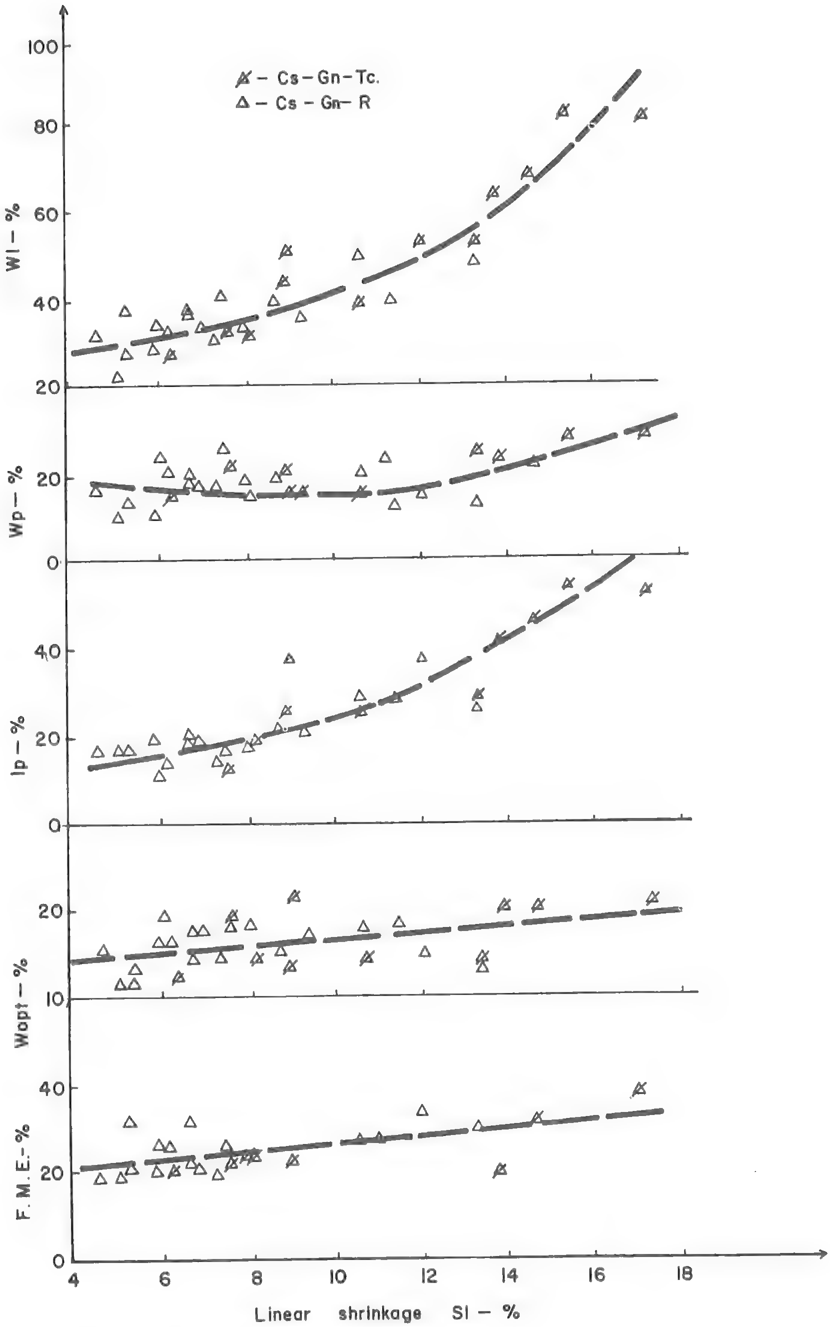


Figure 20. Average curves of the relation between clay content and linear shrinkage.

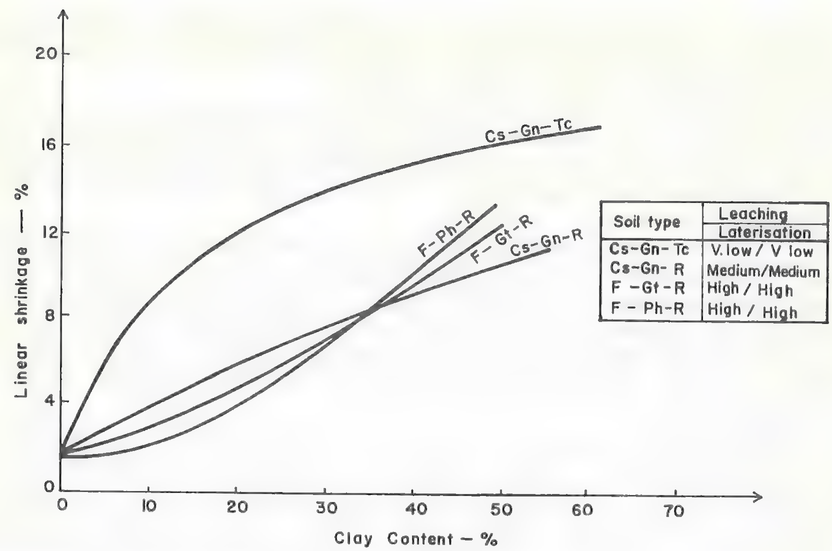


Figure 21. Relation between linear shrinkage and other moisture parameters for coastal savanna zone gneissic soils.

difference was found between the relation for the tropical clays and the other residual soils over gneiss. The effect of laterization does not seem to influence this relationship because the main factor controlling the optimum moisture content is the filling of the soil pores with water and the arrangement of the soil particles. Figure 17 shows the relation between maximum dry density and clay content. It is seen that in general maximum dry densities decrease for all soil groups with increase in clay content.

Specific Gravity—Figure 18 shows the relation between clay content and specific gravity for the soil groups. There was no correlation for the tropical clays and very poor correlation for phyllite soils. Although it is generally believed that all tropical residual soils have rather high specific gravities, it was found that the specific gravity ranged between 2.55 and 2.70. The specific gravities, however, are higher for the laterite gravels in whose fraction laterization is more pronounced. Earlier studies showed that the specific gravity for the laterite gravels is between 2.8 and 3.2, which is attributed to concentration of the iron oxides in the gravel fractions (27).

Shrinkage Properties—Figure 19 shows the relation between clay content and linear shrinkage. A fairly good correlation exists between these two properties. There is also good correlation between linear shrinkage and all the other moisture parameters for each genetic soil group. Figure 20 shows, for example, the relation between linear shrinkage on the one hand and liquid and plastic limits, plasticity index, optimum moisture content (standard Proctor), and field moisture content for the coastal zone gneissic soils. From the average curves in Figure 21 it is seen that the influence of clay content on the linear shrinkage is more pronounced for the tropical clays than for other residual soils. On the basis of a very simple laboratory linear shrinkage test it may be possible to obtain approximate value of other moisture parameters for a genetic soil group.

CONCLUSIONS

The field identification studies carried out by the author and available geological, pedological, and engineering information on the tropically weathered soils of Ghana have revealed that the morphology and general characteristics of these soils are influenced considerably by the weathering systems—the parent material (geological for-

mation), climate, vegetation, topography, and drainage condition—which determine whether the weathering conditions are productive of kaolinite or a montmorillonite type of mineral or some less-known secondary mineral.

Because pedological soil classification is based on the consideration of the major factors of soil formation and because the pedological soil map suggested by Branner and Wills (13) gives the best picture of the distribution of the surface soils, the pedological map is suggested as a sound basis for the initial engineering grouping of Ghanaian subsoils.

The laterization process has a pronounced effect on the chemical and mineralogical composition of Ghana soils. Chemical studies on the soil formations have revealed that the sesquioxide content of the tropical clays and nonresidual soils is generally less than 20 percent, is between 20 and 50 percent for residual soils, and is more than 50 percent for the laterite rocks. The three textural groups of laterite soils (laterite rocks, laterite gravels, and laterite fine-grained soils) reflect definite stages in the process of laterization.

For each genetic fine-grained soil group, consistency parameters, colloidal activity, compaction characteristics, specific gravity, field moisture equivalent, and linear shrinkage have fairly good correlations with the amount of clay fraction content. The different degrees of leaching and laterization of the different soil types in the different climatic-vegetational zones have considerable influence on the relationships between the clay content and soil properties of soils over different parent rocks.

Linear shrinkage has a very good correlation with liquid limit, plastic limit, plasticity index, optimum moisture content, and field moisture equivalent and may be useful as a means of determining these properties for a genetic soil group.

The conclusions reached in the foregoing suggest that the residual fine-grained laterite soils of Ghana may be most adequately classified for highway purposes on the basis of such factors as parent rocks, climatic-vegetational conditions, degrees of weathering (or laterization), and amount of clay content and linear shrinkage.

The conclusions reached, although based on studies in Ghana, may be of practical importance to soils formed under similar conditions in other countries.

It may be of interest to start a basic study on the definition of the degree of laterization in terms of the cementation or increasing thickness of free iron-oxide coating on the soil particles. It would then be possible to investigate its influence on the physicochemical and physical, as well as mechanical, properties of laterite soils. This would considerably improve the present level of knowledge on laterization and properties of laterite soils.

ACKNOWLEDGMENT

The work described in this paper forms part of a long-term study on the engineering classification and utilization of tropically weathered soils. The author wishes to acknowledge with sincere thanks the guidance and encouragement given by Professor Zenon Wilun of State Technical University, Warsaw, who supervised the work. I am also very grateful to Dr. J. W. S. de Graft-Johnson, Director, and Mr. H. S. Bhatia, Head of Soil Mechanics Division, the Building and Road Research Institute, Kumasi, for their useful suggestions.

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REFERENCES

1. Ackroyd, L. W. The Engineering Classification of Some Western Nigerian Soils and Their Qualities in Road Making. Dept. Sci. Ind. Res., British Road Res. Lab., Overseas Bull. No. 10, London, 1959, p. 32.
2. Ackroyd, L. W. The Correlation Between Engineering and Pedological Classification Systems in Western Nigeria and Its Implications. Proc. Third Reg. Conf. for Africa on Soil Mech. and Found. Eng., Salisbury, Vol. 1, 1963, pp. 115-118.

3. Ackroyd, L. W. Formation and Properties of Concretionary and Non-Concretionary Soils in Western Nigeria. Proc. Fourth Reg. Conf. for Africa Soil Mech. and Found. Eng., Cape Town, 1967, pp. 47-52.
4. Ackroyd, L. W. Notes on the Crushing Strength of Some Western Nigeria Concretionary Gravels and Their Selection for Use as a Building Material. Tech. Paper No. 6, Western Nigerian Ministry of Works and Transport, Ibadan, 1960.
5. Ackroyd, L. W. Recommendations for the Construction of Badagry-Ado Road in Western Nigeria. Dept. Sci. Ind. Res., British Road Res. Lab., Research Note RN/3002/LWA, London, 1957.
6. Ackroyd, L. W., and Rhodes, F. G. Notes on the Application of Linear Shrinkage Test to the Selection of Western Nigerian Gravel Soils. Tech. Paper No. 5, Ministry of Works and Transport, Ibadan, 1960.
7. Aitchison, G. D. Soil Morphology and Foundation Engineering. Proc. Third Internat. Conf. Soil Mech. and Found. Eng., Zurich, Vol. 1, 1953, pp. 3-7.
8. Bampo-Addo, A., Raman, K. V., and Mortland, M. M. Clay Mineral Status of Some Major Soil Series in Ghana. Soil Science, Vol. 107, No. 2, 1968, pp. 119-125.
9. Baldovin, G. The Shear Strength of Lateritic Soils. Specialty Session on Engineering Properties of Lateritic Soils. Proc. Seventh Internat. Conf. Soil Mech. and Found. Eng., Mexico City, Vol. 1, 1969, pp. 129-142.
10. Beale, O. W. Dispersion of Lateritic Soils and the Effect of Organic Matter on Mechanical Analysis. Soil Science, Vol. 48, 1939, pp. 475-481.
11. Bates, D. A. Geology of Ghana. In J. B. Wills (Editor), Agricultural and Land Use in Ghana, Oxford Univ. Press, London, 1962, pp. 51-61.
12. British Standard Institution. Methods of Testing Soils for Civil Engineering Purposes. Brit. Std. 1377, London, 1961.
13. Branner, H., and Wills, J. B. Ghana Soils. In J. B. Wills (Editor), Agriculture and Land Use in Ghana, Oxford Univ. Press, London, 1962, pp. 88-126.
14. Buchanan, F. A Journey From Madras Through the Countries of Mysore, Canara and Malabar. East Indian Co., London, Vol. 2, 1807, pp. 436-460.
15. Clare, K. E. Road Making Gravels in Central Africa. British Road Res. Lab., Overseas Bull. No. 12, London, 1960.
16. Clare, K. E. Airfield Construction on Overseas Soils. Part 1. The Formation, Classification and Characteristic of Tropical Soils. Proc. Brit. Inst. of Civ. Engrs., London, Vol. 8, 1957, pp. 211-231.
17. Clare, K. E., and Beaven, P. J. Soils and Other Road Making Materials in Nigeria. Dept. Sci. Ind. Res., British Road Res. Lab., Tech. Paper No. 57, London, 1962, 48 pp.
18. Cooper, G. G. The Bauxites of the Gold Coast (Ghana). Ghana Geological Survey Bull., No. 7, 1936, 33 pp.
19. de Graft-Johnson, J. W. S., Bhatia, H. S., and Gidigasu, M. D. The Engineering Characteristics of Laterite Gravels of Ghana. Specialty Session on Engineering Properties of Lateritic Soils. Proc. Seventh Internat. Conf. Soil Mech. and Found. Eng., Mexico City, Vol. 1, 1969, pp. 117-128.
20. de Graft-Johnson, J. W. S., Bhatia, H. S., and Gidigasu, M. D. The Engineering Characteristics of Lateritic Residual Clay of Ghana for Earth Dam Construction. Proc. Symposium on Rock and Earth Fill Dams, New Delhi, Vol. 1, 1968, pp. 94-107.
21. de Graft-Johnson, J. W. S., and Bhatia, H. S. Engineering Characteristics of Laterite Soils (General Report). Specialty Session on Engineering Properties of Laterite Soils. Seventh Internat. Conf. Soil Mech. and Found. Eng., Mexico City, 1969, 20 pp.
22. de Graft-Johnson, J. W. S., Bhatia, H. S., and Gidigasu, M. D. The Strength Characteristics of Residual Micaceous Soils and Their Application to Stability Problems. Proc. Seventh Internat. Conf. Soil Mech. and Found. Eng., Mexico City, Vol. 1, 1969, pp. 165-172.
23. Dowling, J. W. F. The Occurrence of Laterites in Northern Nigeria and Their Appearance in Aerial Photography. Eng. Geol., Vol. 1, No. 3, 1966, pp. 221-233.

24. Dumbleton, N. J., West, C., and Newill, U. The Mode of Formation, Mineralogy and Properties of Some Jamaican Soils. Eng. Geol., No. 3, 1966, pp. 235-249.
25. Easterbrook, P. L. The Preliminary Selection of Road Base Materials in Western Nigeria by Linear Shrinkage. Tech. Paper No. 9, Ministry of Works and Transport, Ibadan, 1962.
26. Fermor, L. L. What Is Laterite? Geolog. Mag., Vol. 5, No. 8, 1911.
27. Gidigasu, M. D. The Highway Geotechnical Properties of Ghana Soils. PhD thesis, State Technical Univ., Warsaw, 1969, 171 pp.
28. Gidigasu, M. D. The Engineering Characteristics of Laterite Soils (Literature Review). Project Rept. No. 7, Building and Road Research Institute, Kumasi, Ghana, 1970, 92 pp.
29. Hamilton, R. Minerals in the Gold Coast (Ghana) Soils. Jour. of Soil Science, Vol. 6, 1955, pp. 312-318.
30. Hamilton, R. Microscopic Studies of Laterite Formation in Ghana. Proc. Second Internat. Working Meeting on Soil Micromorphology, Arulen, Netherlands, 1964, pp. 269-278.
31. Hogentogler, C. A. Engineering Properties of Soils. McGraw-Hill Book Co., New York, 1937.
32. Hunt, C. B. Some Geological Aspects of the Problems of Soil Genesis and Soil Classification. Proc. Second Internat. Conf. Soil Mech. and Found. Eng., Rotterdam, Vol. 3, 1948, pp. 6-7.
33. Joachin, A. W. R., and Kandiah, S. The Composition of Some Local Laterites, Soil Concretions and Clays. Trop. Agriculturist, No. 96, 1941, pp. 67-75.
34. Janny, H. Factors of Soil Formation. McGraw-Hill Book Co., New York, 1941, 281 pp.
35. Kerbyson, J. D., and Schandorf, J. R. H. Chemical Analyses of Ghana Rocks, Ores and Minerals. Ghana Geological Survey Bull. No. 42, 1966, 166 pp.
36. Lacroix, A. Les laterites de la Guinée et les produits d'alteration qui leur sont associés. Nouv. Arch. Mus. Hist. Nat., Vol. 5, 1913.
37. Little, A. D. Laterites. Proc. Third Asian Regional Conf. Soil Mech. and Found. Eng., Haifa, Vol. 11, 1967, pp. 61-71.
38. Little, A. L. The Engineering Classification of Residual Tropical Soils. Specialty Session on Engineering Properties of Lateritic Soils. Proc. Seventh Internat. Conf. Soil Mech. and Found. Eng., Mexico City, Vol. 1, 1969, pp. 1-10.
39. Lumb, P. The Properties of Decomposed Granite. Geotechnique, Vol. 12, No. 3, 1962, pp. 226-243.
40. Lumb, P. Effect of Rainstorm on Slope Stability. Symp. on Hong Kong Soils, Hong Kong Group, 1962, pp. 73-87.
41. Martin, F. J., and Doyne, H. F. Laterites and Lateritic Soils in Sierra Leone. Jour. Agric. Sci., Vol. 17, pp. 530-546; Vol. 20, pp. 135-143, 1927.
42. Moh, Z. C., and Mashar, F. M. The Effects of Method of Preparation on the Index Properties of Lateritic Soils. Specialty Session on Engineering Properties of Lateritic Soils. Proc. Seventh Internat. Conf. Soil Mech. and Found. Eng., Mexico City, Vol. 1, 1969, pp. 23-35.
43. Newill, D. An Investigation of the Linear Shrinkage Test to Tropical Soils and Its Relation to Plasticity. D. S. I. R., Road Res. Lab., Research Note No. RN/4106/DN, 1961.
44. Novais-Ferreira, H., and Correia, J. A. The Hardness of Laterite Concretions and Its Influence in the Performance of Soil Mechanics Tests. Proc. Sixth Internat. Conf. Soil Mech. and Found. Eng., Montreal, Vol. 1, 1965, pp. 82-86.
45. Pendleton, R. L., and Sharasuvana, S. Analyses of Some Siamese Laterites. Soil Science, Vol. 62, 1946, pp. 423-440.
46. Remillon, A. Les recherches routieres entreprise en Afrique d'expression francaise. Annale de I. T. B. T. P., No. 231-232, 1967, pp. 366-388.
47. Ruddock, E. C. Residual Soils of the Kumasi District. Geotechnique, Vol. 17, No. 4, 1967, pp. 359-377.

48. Sherwood, P. T. Classification Tests on African Red Clays and Keuper Marl. Quarterly Jour. of Eng. Geol., London, Vol. 1, No. 1, 1967, pp. 47-55.
49. Skempton, A. W. The Colloidal "Activity" of Clays. Proc. Third Internat. Conf. Soil Mech. and Found. Eng., Zurich, Vol. 1, 1953, pp. 57-61.
50. Stephen, L. A Petrological Study of Tropical Black Earth and Grey Earth From the Gold Coast (Ghana). Jour. of Soil Science, Vol. 4, 1953, pp. 211-219.
51. Terzaghi, K. Design and Performance of the Sasumua Dam. Jour. Inst. Civ. Engrs., London, Vol. 9, 1958, pp. 369-394.
52. Walker, H. O. Weather and Climate of Ghana. In J. B. Wills (Editor), Agriculture and Land Use in Ghana, Oxford Univ. Press, 1962, pp. 7-50.
53. Winterkorn, H. F. Discussion on Soil Properties as Function of General Soil Forming Processes. Proc. First Internat. Conf. Soil Mech. and Found. Eng., Cambridge, Mass., Vol. 3, 1936.
54. Winterkorn, H. F. Engineering Uses and Limitations of Pedology for Regional Exploration of Soils. Proc. Second Internat. Conf. Soil Mech. and Found. Eng., Rotterdam, Vol. 1, 1948, pp. 8-12.
55. Wooltorton, F. L. D. Discussion on Residual Soils of Kumasi District in Ghana. Geotechnique, Vol. 18, No. 2, 1968.
56. Yeboah, S. L., and Hornsby-Odoi, A. G. The Effect of Pretreatment on the Atterberg's Limits of Laterite Soils. Project Rept. SM No. 4, Building and Road Research Institute, Kumasi, Ghana, 1970.
57. Youssef, M. S., Sabry, A. E., and Ramil, A. H. Temperature Changes and Their Effects on Some Physical Properties of Soils. Proc. Fifth Internat. Conf. Soil Mech. and Found. Eng., Paris, Vol. 1, 1961, pp. 419-421.
58. Soil Survey Technical Reports, Nos. 26, 38, 41, 44, 52, 54, 56, 62. Ministry of Agriculture, Ghana.

THE INFLUENCE OF SESQUIOXIDES ON LATERITIC SOIL PROPERTIES

Frank C. Townsend, Soils Research Division, Waterways Experiment Station, Vicksburg, Mississippi; and
Phillip G. Manke and James V. Parcher, School of Civil Engineering, Oklahoma State University

The breakdown of the characteristic granular structure of lateritic soils by remolding is known to alter the plastic, textural, and engineering characteristics of these soils. This laboratory study was an investigation of some mineralogical, chemical, physical, and engineering properties of a lateritic soil in order to evaluate more fully the basic reasons for the effects of remolding. Results of X-ray diffraction and scanning electron microscopy studies showed that the hydrated iron and aluminum oxides (sesquioxides) coat the clayey constituents of the soil and bind them into coarser micro-aggregations. Analyses of grain size indicated that the remolding phenomenon disaggregates the micro-aggregations into finer clayey clusters because of the friable nature of the sesquioxidic cementing agents. DTA and chemical analyses revealed the presence of substantial quantities of amorphous material in the soil. Atterberg limits tests on sesquioxide-free soil indicated that the abnormally high moisture contents of this lateritic soil are a result of the high moisture-retention capabilities of the amorphous colloids and the porous micro-aggregations in the soil. Although remolding greatly altered the index properties, little change was observed in the strength characteristics of the various soil states. The magnitude of the angles of internal friction determined by triaxial compression tests reflect a greater amount of interlocking by the micro-aggregations than would be produced by a platey structure. Unconfined compression tests showed that lime was an effective stabilizer for this lateritic soil.

•LATERITE and "lateritic soils" are terms used to describe the red residual soils that occur abundantly throughout the tropical and subtropical regions of the world. Geomorphologically, laterization involves the leaching out or removal of the silica, alkali, and alkaline earths and the concentration of hydrated iron and aluminum oxides (hereafter referred to as "sesquioxides"). Generally, the resulting soil in situ possesses a granular structure due to these sesquioxides, which coat and knit the indigenous soil particles into tiny spherical aggregations (1).

Previous studies (16, 21, 23, 24) indicated that these micro-aggregations are quite friable and are readily disaggregated by remolding of the soil. Depending on the amount of remolding, this behavior produces considerable variations in the plasticity, grain size distribution, and engineering characteristics of these soils. As a result, there is no assurance that index tests can be used for predicting the engineering behavior of these soils, and such results reported in the literature may be unreliable. Consequently, the engineering classification of tropical-climate clays by systems oriented for temperate-climate clays has been questioned. Likewise, remolding has been shown to exert a considerable influence on the response of the soil to various stabilizing additives.

In a previous paper (23) the authors presented quantitative differences in the plasticity, textural characteristics, and stabilization responses of a relatively undisturbed or

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unremolded (previously referred to as unworked) and remolded lateritic soil. These differences were attributed to a breakdown of the granular micro-aggregations in the soil. In that paper, it was postulated that a substantial portion of the clay particles had been coated with sesquioxide films and that working or remolding by mechanical agents caused a breakdown of the aggregations, partially abraded these films, and resulted in a more characteristic behavior of the clay particles.

Bennett (3) observed that the physical properties of clay colloids are influenced by their silica-sesquioxide ratio ($SiO_2R_2O_3$). He concluded that on the basis of the SiO_2/R_2O_3 ratio a differentiation might be made between friable nonplastic soils and nonfriable plastic soils. Such a differentiation would be a considerable asset to highway engineers in determining the quality and suitability of a doubtful clay soil in tropical areas for use as a road or airfield subgrade. Likewise, Winterkorn (24) commented that "the presence of iron in lateritic soils is one of the most important factors which influences their engineering properties." Based on these concepts and the authors' previous postulation, it appears that the sesquioxides and their relationship with the indigenous soil particles are key factors in understanding the usual behavioral characteristics of lateritic soils. The present investigation was directed toward studying the influences of the sesquioxides on the Atterberg limits, grain size distribution, and lime susceptibility of a Panamanian lateritic soil. These influences were evaluated by chemically extracting the sesquioxides, testing the sesquioxide-free soil, and comparing the test results with those of the previous study (23).

RELATIONSHIP BETWEEN CLAYS AND SESQUIOXIDES

Unusual soil behavior can frequently be explained in terms of composition (particularly the clay mineralogy) and soil structure. The clay minerals encountered in lateritic soils are principally 1:1 minerals, of which kaolinite is by far the most common. Halloysite occurs less frequently but is known to exist in lateritic soils. Montmorillonite and other 2:1 clay minerals generally are not associated with lateritic soils. The sesquioxides may exist in a variety of forms, of which goethite and hematite and gibbsite and boehmite respectively are the most common forms of these iron and aluminum oxides. Various amorphous forms of hydrated iron and aluminum oxides, or sesquioxidic allophane, are also known to exist in these soils (4, 10, 12).

Many investigators have shown that the sesquioxides are adsorbed on the surfaces of the clay minerals in lateritic soils. This adsorption occurs through the interaction of the positively charged sesquioxides and the negatively charged clay particles. However, the mechanisms governing the adsorption are quite complex and dependent on the surface characteristics of the iron and aluminum compounds, the charge characteristic of the clay minerals, and the pH of the surrounding medium (6). Mattson (13) demonstrated that the sesquioxides behave as electrical ampholytes, i. e., above a pH of 7.0 they are electronegative, whereas below this pH level they are electropositive. The adsorption of iron oxides on the surfaces of clay minerals must, therefore, occur at pH values below 7.0. Greenland and Oades (6), using electron microscropy, observed that at low pH values iron precipitates as a surface coating on kaolinite and that some of these surfaces are bound together by the iron hydroxide. Electron microscopy studies by Follett (5) indicated that colloidal iron hydroxide is adsorbed only on one basal surface of the kaolinite flakes. Apparently only the silica tetrahedral surfaces possess sufficient charge to fix the iron particles. The nature of the alignment of iron particles, according to Follett, suggested some neutralization of the charge on the colloidal iron that resulted in aggregation of the clays.

Petrographic studies (1, 7, 9, 18, 19, 20) reveal that lateritic soils often possess a porous granular structure consisting of iron-impregnated clayey material in minute spherical aggregations resembling popcorn balls. The strength of these aggregations is derived from the thin ferruginous films found within the microjoints of the elementary clay particles and as coatings over the particles (18). The role of the sesquioxides in the formation of these micro-aggregates is generally ascribed to one or all of the following: (a) cementation due to precipitation of a hydrated iron or aluminum gel and a subsequent irreversible dehydration of these materials, (b) the presence of iron in solution,

which prevents deflocculation, and (c) the formation of organic mineral compounds of humic acids with free sesquioxides (14).

INVESTIGATIVE PROCEDURES

Materials

Soil—The lateritic soil used in this investigation was obtained from a borrow pit located in Curundu, Panama Canal Zone. The geological profile of the borrow pit has been described elsewhere (23).

Lime—The lime used in this study is a quicklime (CaO) obtained from the St. Clair Lime Company, Sallisaw, Oklahoma. Because of the unstable behavior of quicklime upon exposure to the atmosphere, fresh lime samples were used to avoid carbonated fractions.

Sample Preparation

Sesquioxide Removal—The free iron and aluminum oxides in the soil were extracted by the sodium dithionite-citrate-bicarbonate treatment of Mehra and Jackson (15).

Remolding—Remolding of the soil to simulate the effects of heavy construction equipment was accomplished by mixing the soil in a Hobart mixer at a water content above the liquid limit for 2 hours. Unremolded soil was obtained by gently hand sieving the material at low moisture contents. Further details have been presented elsewhere (23).

Compaction—All test specimens were impact-compacted in a Harvard miniature compaction apparatus. For an objective comparison of the strength values of various soil-lime mixtures, it is essential that all specimens be compacted to the same density and possess relatively the same particle orientation. These criteria were developed in previous studies (23).

Methods and Equipment

X-Ray Diffraction—The mineralogical composition of the soil fractions was analyzed by X-ray examination of the oriented specimens (10). The procedure that was used involved separation, sesquioxide removal, and ion saturation. The silt and clay fractions were separated by gravity sedimentation. The coarse and fine clay fractions were subdivided by using a Sharples Supercentrifuge. Potassium or calcium saturations were accomplished by three washings with 1 N KCl or 1 N $CaCl_2$ respectively. The X-ray diffraction patterns were obtained by using a General Electric XRD 6 and a SPG 2 spectrogoniometer with a Ni filtered Cu Kα radiation generated at 40 KV and 20 ma. The slits used were 1 deg MR beam and HR sollar and 0.2 deg detector slit. The settings were T. C. of 2.0, CPS range of 1,000, and amplitude gains of 16 for coarse and 88 for fine.

Differential Thermal Analysis—The soil material analyzed consisted of unremolded, remolded, and "sesquioxide-free" soil samples that passed the No. 80 U. S. Standard sieve. These samples were allowed to equilibrate for 4 days in a sealed chamber over a saturated solution of magnesium nitrate, $Mg(NO_3)_2 \cdot 6H_2O$. This pretreatment assured an exposure of the samples to a standard of 56 percent relative humidity, which permitted valid comparisons of the low-temperature peak system of clay thermograms (11). The DTA curves were obtained by using a Fisher Differential Thermalyzer Model 260 connected to a 1-mv strip chart recorder. Operating procedures involved heating the sample from ambient temperature to 1,200 C at a rate of 10 deg per minute.

Grain Size Distribution—The hydrometer method of analysis was used to obtain the grain size distribution curves of the portion passing the No. 100 U. S. Standard sieve. The unremolded soil was dispersed by gentle rotation of the graduated cylinder, while the remolded soil was dispersed in an electric mixing cup according to the usual procedure. Subsequently, the sesquioxides were extracted from the soil samples used for the unremolded and remolded analyses, and the grain size distributions of the sesquioxide-free soils were determined. Two milliliters of a 4 percent solution of Calgon were used as the deflocculating agent for all hydrometer tests.

Scanning Electron Microscopy—The scanning electron micrographs of the remolded, remolded plus lime, and sesquioxide-free soils were taken with a JEOLCO JSM-2

scanning electron microscope. Prior to scanning, the samples were coated by vaporizing a gold-palladium alloy metal wire in a JEOLCO JEE-4C vacuum evaporator.

DISCUSSION OF RESULTS

Mineralogical Analysis

The X-ray diffraction patterns of the coarse clay fractions of sesiquoxide-free and remolded soil samples shown in Figure 1 reveal that the clay mineral of this soil is a 1:1 poorly crystallized kaolinite. The existence of iron coatings on the soil particles is indicated in this figure by the occurrence of a previously absent quartz peak (3.35 Å) after removal of the sesquioxides. The increased sharpness of the kaolin peaks (7.24 Å and 3.47 Å) upon treatment of the soil for sesquioxide removal also substantiates the presence of iron coatings. Additional X-ray diffraction studies of the silt fractions of the same samples shown in Figure 2 reveal that the soil has been extremely weathered with only some resistant quartz grains, a poorly crystallized kaolin clay, and magnetite remaining as the identifiable minerals. The presence of some secondary silica as cristobalite and some amorphous material is also suspected. The iron coatings inferred from comparisons of these diffraction patterns must exist as microcrystals or in amorphous forms that are undetectable by X-ray. No peaks were observed for goethite (4.15 Å, 2.69 Å, and 2.44 Å) or hematite (2.69 Å, 2.57 Å, and 1.69Å), which are the most common forms of iron oxide coatings in soils. These findings are consistent with those of other investigators (8) who identified quartz, "intermediate kaolinite" (poorly crystallized kaolinite), and crystobalite as the mineral constituents of Panamanian lateritic soils.

The DTA thermograms of the unremolded, remolded, and sesquioxide-free samples are shown in Figure 3. These patterns indicate the presence of an amorphous material that is undetectable by X-ray diffraction. The large endothermic peak at 100 C is an important feature of thermograms for halloysite and allophane because kaolinite only exhibits a relatively small peak (10, 11). Allophane, an amorphous hydrous aluminosilicate, often occurs as a co-precipitate of iron oxides in highly weathered latosols and is believed to be a weathering relic (10) or precursor (1, 8) of halloysite or kaolinite. The marked decrease of this 100 C temperature endotherm in the iron-free sample

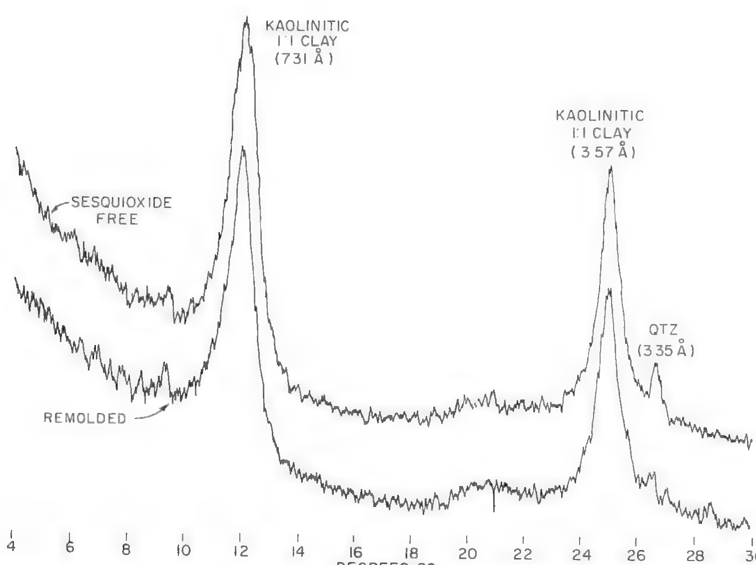

Figure 1. X-ray diffraction patterns of the coarse clay fraction (K-saturated).

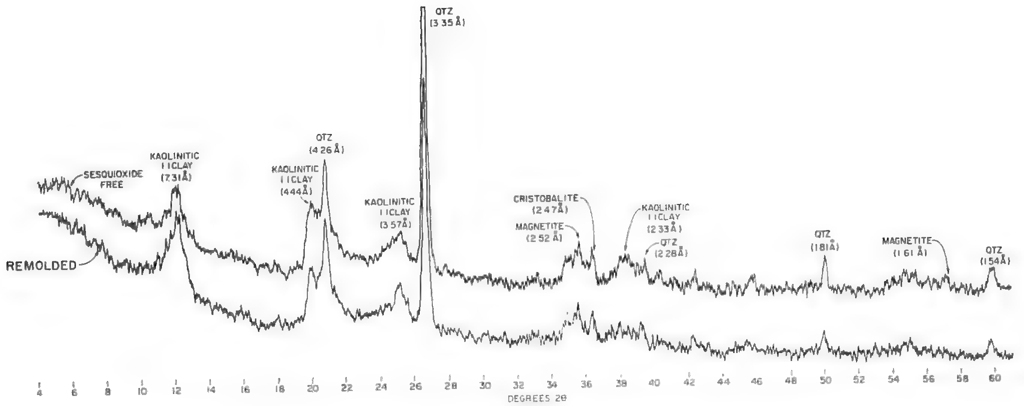

Figure 2. X-ray diffraction patterns of the silt fractions (Ca-saturated).

indicates the presence of an allophanic material. If this endotherm had been attributed solely to halloysite, it would still exist at an appreciable magnitude after the iron-removal treatment.

A large endotherm at 500 to 600 C is characteristic of 1:1 kaolin minerals, although highly crystallized kaolinite generally exhibits an endothermic peak at 600 C. However, a poorly crystallized kaolinite will dehydroxylate at slightly lower temperatures. The symmetry of this peak discredits the presence of halloysite. Thus, the main endotherm at 560 C confirms the X-ray diagnosis that the kaolinite is poorly crystallized.

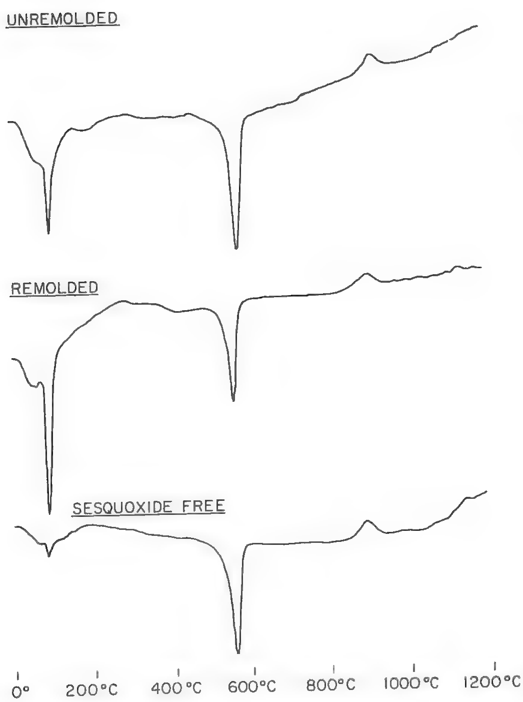

Figure 3. DTA thermograms of unremolded, remolded, and sesquioxide-free lateritic soil.

The small exotherm at 880 C is suggestive of a mixture of poorly crystallized 1:1 kaolin minerals. The exothermic peak for highly crystallized kaolinite generally occurs in the neighborhood of 950 to 980 C. However, if the kaolinite is poorly crystallized or if allophane is present, this exotherm will be lower (10, 11).

The presence of a considerable quantity of amorphous material in the soil, perhaps as sesquioxidic allophane, was confirmed by a chemical analysis for allophane (10). The analysis revealed that approximately 20 percent free SiO_2, 4 percent free Al_2O_3, and 6 percent Fe_2O_3 exists in the minus No. 40 U.S. Standard sieve fraction. Generally, lateritic soils are considered to be low in free silica and relatively inactive. However, amorphous aluminosilicates have been reported in Panamanian soils (4) and in soils derived by the weathering of andesite (8), one of the parent rocks for this particular lateritic soil. The presence of these amorphous colloids undoubtedly contributes greatly to the behavior of this soil, as high water-retention capabilities and large specific surfaces are characteristic of amorphous colloids.

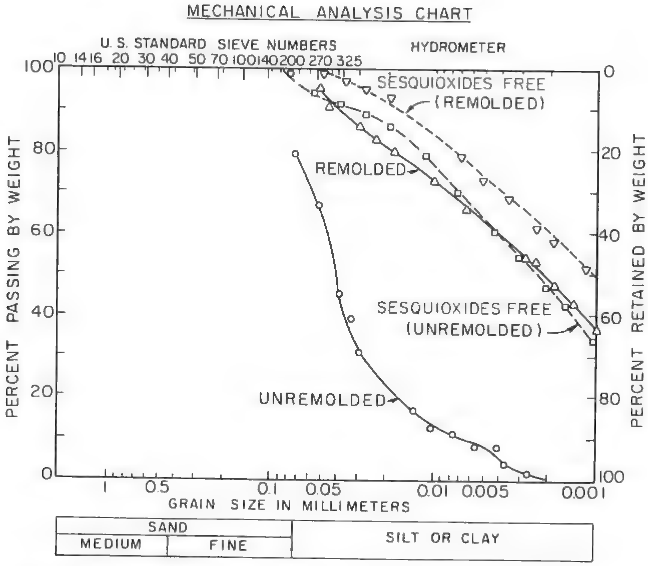

Figure 4. Effect of sesquioxide removal on the grain size of lateritic soil.

Physical Properties

Grain Size Analysis—Grain size distribution curves in Figure 4 show that remolded soil has a greater percentage of fine particles than does unremolded soil. This greater percentage of fines indicates that remolding by mechanical agents causes disaggregation of the friable granular soil aggregates. A comparison between the grain size curves for the unremolded soil in the natural and sesquioxide-free conditions shows a considerable increase in finer sizes after removal of the sesquioxides. This increase confirms that the soil in its natural state consists of clay-sized particles bonded into micro-aggregates by sesquioxide cementing agents. Similar results have been reported by Winterkorn (24) and Pearring (17), who showed increases in clay-size fractions and corresponding decreases in sand- and silt-size fractions after removal of the sesquioxides from various lateritic soils.

A comparison of the grain size curves for remolded soil in the natural and sesquioxide-free conditions reveals that removal of the sesquioxides further increases the amount of fines in the soil. This increase suggests that the iron and aluminum oxides continue to coat and to bond the soil particles. Thus, the remolding phenomenon merely produces smaller micro-aggregates of iron-impregnated clays.

TABLE 1

PHYSICAL PROPERTIES OF UNREMOLDED, REMOLDED, AND SESQUIOXIDE-FREE LATERITIC SOIL

Property	Unremolded	Remolded	Sesquioxide-Free
Atterberg limits			
Liquid limit (percent)	57.8	69.0	51.3
Plastic limit (percent)	39.5	40.1	32.1
Plasticity index (percent)	18.3	28.9	19.2
Specific gravity	2.80	2.80	2.67
Proctor density (pcf)	84.5	83.0	88.0
Optimum moisture content (percent)	35.0	34.5	29.5

Atterberg Limits—The results of the Atterberg limits tests given in Table 1 show that remolding increased the liquid limit of the soil, whereas the plastic limit remained essentially the same. This increase in plasticity has been attributed to the breakdown of the granular structure, which increases the amount of fines and exposes more surface area for the adsorption of moisture (16, 23, 24).

Removal of the iron and aluminum oxides would be expected to increase the plasticity of the soil because of (a) increased amounts of fine particles released from the previously cemented clay clusters and (b) removal of the plasticity-suppressing sesquioxide coatings from the surfaces of the clay minerals. Newill (16) obtained an increase in liquid limit from 77 percent to 93 percent after removal of the iron oxides from his lateritic soil sample. However, the results given in Table 1 show that removal of the sesquioxides caused a significant decrease in the liquid limit and plastic limit values of the soil. Apparently, removal of the sesquioxides alters the water-retention capabilities of the soil. This alteration was also shown by the drastic reduction in peak intensity of the 100 C peak of the DTA curves in Figure 3. One of the characteristics of amorphous colloids is a high water-retention capability due to their large specific surfaces. Therefore, the decrease in plasticity of the remolded soil from 28.9 to 19.2 percent after removal of the sesquioxides can be partially attributed to the removal of these amorphous sesquioxides of the soil. Concurrently, removal of sesquioxidic cementing agents destroys the porous micro-aggregates, thereby eliminating the micro-voids and clayey clusters that Terzaghi (21) described as the cause for the high Atterberg limits associated with lateritic soils.

Engineering Properties

Proctor Density—Density values were determined from miniature samples compacted with an energy per unit volume equal to that used in the standard Proctor compaction test. These values are given in Table 1. Usually, rather high compacted densities would be anticipated for lateritic soils because of the high specific gravity of the abundant iron oxides (2). However, compaction studies revealed very low densities and very high optimum moisture contents in comparison with temperate-zone clays with similar index values. The low density of this soil, 84.5 pcf, is attributed to its granular nature. The micro-aggregates in the soil impart a high void ratio and corresponding low density. The relatively high optimum moisture content results from water held within the porous micro-aggregations and from that associated with the amorphous constituents of the soil. This moisture is trapped and unable to contribute substantially to the "lubrication" of the soil particles during compaction.

Contrary to normal expectations, removal of the heavy sesquioxides resulted in an increase in density (84.5 to 88.0 pcf) and a decrease in optimum moisture content (35.0 to 29 percent). This apparently contradictory behavior is attributed to the breakdown of the granular structure by removal of the sesquioxidic cementing agents. This breakdown allows a closer packing of the clayey particles and a lower void ratio. The decrease in optimum moisture content is due to an alteration of the amorphous colloids and porous micro-aggregates. The effects are analogous to those resulting in the reduction in plasticity.

Shear Strength Parameters—The results of various triaxial compression tests for compacted soil in the saturated and unsaturated conditions are given in Table 2. The undrained test (Q) was used to evaluate ϕ and c values of unsaturated soil, whereas the "effective stress" parameter, ϕ', was measured by consolidated-undrained tests with pore pressure measurements ($\overline{R}$). These results indicate that, despite its low density and high plasticity, this soil possesses satisfactory strength characteristics in a compacted state. The magnitude of the ϕ' values reflects the inherent granular nature of the soil.

Apparently, remolding has little effect on the shear strength parameters of the compacted soil. However, remolding decreased the angle of internal friction and increased the cohesion. This reduction in ϕ and ϕ' is attributed to a reduced interlocking of the disaggregated soil grains, whereas the increase in cohesion is due

TABLE 2

SHEAR STRENGTH PARAMETERS FOR A LATERITIC SOIL

Test Conditions	Unremolded	Remolded	Sesquioxide-Free
Undrained test (Q)			
ϕ (deg)	26.0	24.0	11
c (kg/cm^2)	0.42	0.56	0.91
Consolidated-undrained ($\overline{R}$)			
ϕ' (deg)	38	37	32

to a more characteristic behavior of the disaggregated clays. Nevertheless it must be concluded that, although remolding greatly influences the index properties, little change is observed in the strength characteristics of the various soil states.

Lime Stabilization—The results of lime stabilization are shown in Figure 5. For these tests 5 percent lime was selected as the comparison percentage. Comparisons of the various curves shown in Figure 5 reveal some interesting aspects. The unremolded, remolded, and sesquioxide-free soils all exhibit strength increases with time. These strength increases indicate that pozzolanic reactions are occurring. However, the variations in the magnitude and the speed of the strength gains suggests that the susceptibility of the soil to stabilization is influenced by mechanical working and chemical alteration.

Because most or all of the free silica has been removed from lateritic soils during the genesis of the soil, the primary source of silica available for pozzolanic reactions would be in the inherent clay minerals. However, the presence of sesquioxidic coatings on the surfaces of these clays inhibits the calcium and clay (silica) reactions. Thus, the retarded and poor response of the lime-stabilized unremolded soil in producing significant strength increases is attributed to the nonavailability of the clay minerals for reaction with the lime. The exhibited strength gains of the unremolded soil are probably limited to the reaction of the lime with small amounts of amorphous silica.

Although the clay surfaces probably remain coated by the sesquioxides after mechanical remolding, evidently the lime susceptibility of the soil is enhanced. Apparently the increased amount of finer particles produced by remolding supplies greater surface areas for soil-lime reactions. Therefore, the primary difference between the responses of the unremolded and remolded soil to lime treatments is attributed to particle size.

Generally, it would be expected that removal of the sesquioxides would produce greater strength gains than those obtained for the natural soil in the unremolded and remolded conditions. These greater strengths would result from (a) increased amount of fines due to the chemical breakdown of the clay aggregations and (b) removal of the protective sesquioxidic coatings from the surfaces of the indigenous clay particles. Thompson (22) evaluated this concept using natural and iron-free non-lateritic soils that were treated with 5 percent lime. His results showed that strength increases were greater for the soil in the iron-free state than for the corresponding natural condition. However, Figure 5 shows that removal of the sesquioxides from the Panamanian lateritic soil produced strengths higher than those of the unremolded soil but lower than those of the remolded soil. Thus, the concept that the free iron and aluminum oxides retard the lime susceptibility of the soil is only partially substantiated.

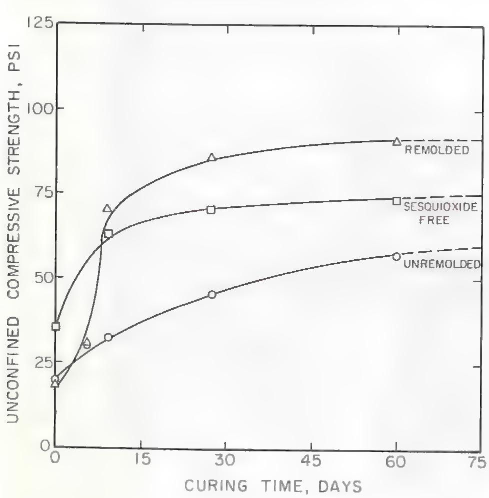

Figure 5. Effect of curing age on unconfined compressive strength of a lateritic soil plus 5 percent lime.

Comparisons of the strengths of remolded soil with those of sesquioxide-free soil indicate that extraction of the sesquioxides reduces the lime reactivity of the soil. Because both the remolded and sesquioxide-free samples possess similar grain size distributions (Fig. 4), it appears that the sesquioxide extraction process removed something that contributed to the stabilizing reactions. Previously it was indicated that the extraction procedure greatly alters the characteristics of the amorphous materials in the soil. Therefore, it is apparent that the major contributors to the stabilizing reactions in this soil are the amorphous alumina and silica because these amorphous colloids can readily enter into pozzolanic reactions because of their high specific surfaces and reactivity. It appears reasonable to conclude that lime

would be an ineffective stabilizer for lateritic soils unless a substantial quantity of amorphous allophanic colloids are available in the soil for pozzolanic reactions.

The scanning electron micrographs shown in Figures 6 through 9 were obtained using samples from sealed lime-treated, remolded specimens that had cured for more than 18 months in a moist room at 20 C. The micrographs were made to observe the various types of soil-lime reaction products and to determine at what lime concentration they were formed. Generally, these reaction products have been identified as hydrated forms of calcium silicate (CSH) that resemble the tobermorite minerals and/or calcium aluminate (CAH).

Photomicrographs of remolded and sesquioxide-free specimens to which no lime has been added are shown in Figures 6 and 7 respectively. The granular, fluffy, micro-aggregated clays of the remolded soil are easily seen in Figure 6. Contrasting with this aggregated structure is the platey structure of the sesquioxide-free soil shown in Figure 7. Jointly, these two photomicrographs strikingly show the agglomerating effect of the sesquioxides.

A photomicrograph of a specimen to which 10 percent lime has been added to the remolded soil is shown in Figure 8. The formation of new crystals is quite evident in this figure. The irregular platelets whose edges appear as laths are tentatively identified as CSH. The cubic crystals (grape-like bunches) shown in Figures 8 and 9 are thought to be lime, CaO, that has not entered into any pozzolanic reactions. However, CAH also possesses a cubic structure, and these cubes may be some form of this compound.

A photomicrograph of a specimen to which 20 percent lime has been added to the remolded soil is shown in Figure 9. The rolled-up tubular sheet is tentatively identified as a pseudo-tobermorite mineral (CSH).

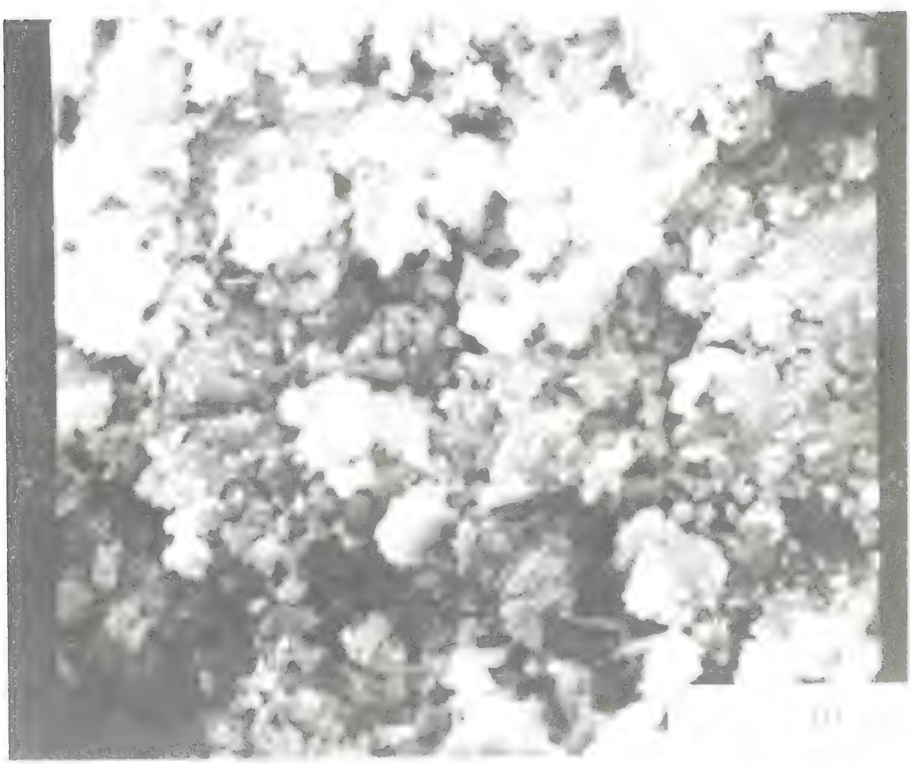

Figure 6. Scanning electron photomicrograph of remolded lateritic soil (3,000X).

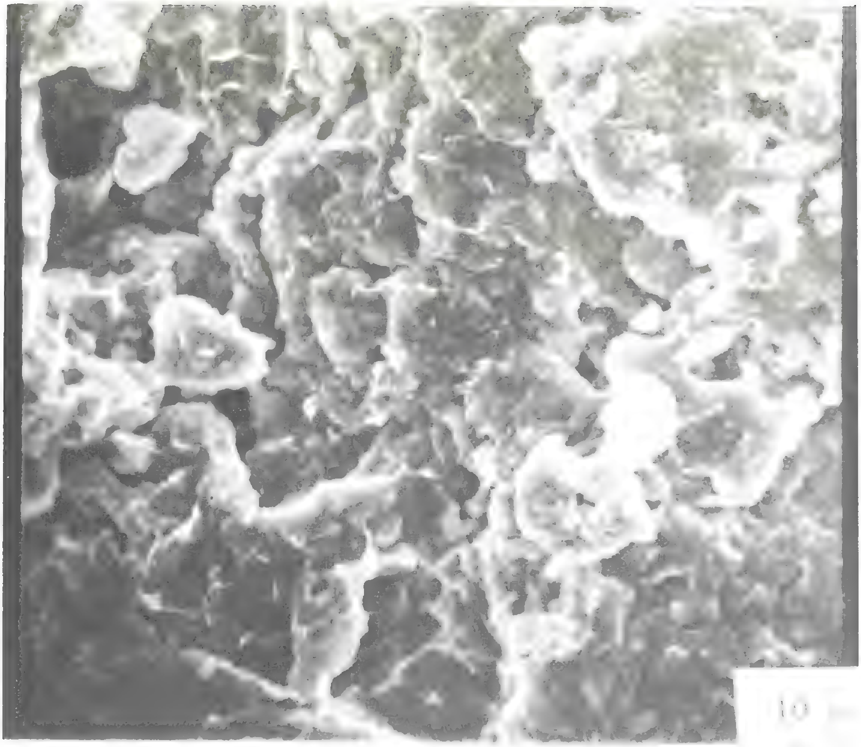

Figure 7. Scanning electron photomicrograph of sesquioxide-free lateritic soil (600X).

Figure 8. Scanning electron photomicrograph of remolded lateritic soil plus 10 percent lime (600X).

Figure 9. Scanning electron photomicrograph of remolded lateritic soil plus 20 percent lime (2,400X).

Figures 8 and 9 show that lime reaction crystals are formed in the lateritic soil-lime mixtures. However, these crystals were only observed in photomicrographs of soil-lime mixtures containing more than 5 percent lime.

CONCLUSIONS

The results of this investigation revealed that the sesquioxides of iron and aluminum strongly influence the engineering behavior of this lateritic soil. The following conclusions can be made for this type of soil and the testing procedures employed:

1. Mineralogically, the soil is composed primarily of remnant quartz grains and a 1:1 poorly crystallized kaolinite. Secondary silica, such as cristobalite and chalcedony, and magnetite exist in the soil in lesser amounts. In addition to these minerals, considerable quantities of free amorphous iron, aluminum, and silica are present in the soil.
2. The sesquioxides of iron and aluminum influence the behavior of this soil by coating the clayey constituents of the soil and binding them into coarser aggregations. These sesquioxidic coatings on the indigenous clay minerals suppress their normal behavioral characteristics, and the aggregations impart a granular structure to the soil.
3. The amorphous allophanic constituents are largely responsible for the exhibited physicochemical behavior of the soil because of the large specific surfaces and high moisture-retention capabilities characteristic of these materials. Undoubtedly the index properties and chemical stabilization susceptibility of the soil would be affected by the presence of these colloids.
4. Remolding of the soil greatly influences the textural characteristics and plasticity. The granular structure is quite friable and readily breaks down to increase the percentage of finer particles. The increased amounts of fines cause an increase in the plas-

ticity of the soil. The high moisture-retention capabilities of the amorphous constituents provide abnormally high moisture contents for this soil. Because these index properties are affected by remolding and the presence of amorphous colloids, new or revised laboratory tests should be developed to evaluate the properties of lateritic soils. The SiO_2/R_2O_3 ratio provides a possible means of predicting the engineering characteristics of lateritic soils.

5. The inherent granular structure of the soil influences the engineering characteristics. Despite the presence of heavy sesquioxides in the soil, the granular aggregations produce a soil that has a low density and high void ratio. The high angles of internal friction reflect a greater amount of interlocking than is normally found in soils having such a high proportion of platey minerals. Disruption of the granular structure by remolding has little effect on the shear strength parameters of the compacted soil.

6. Lime is an effective stabilizer for this lateritic soil. Its effectiveness is primarily attributed to the availability of amorphous silica, rather than to the sesquioxide-coated clay minerals, for strength-producing pozzolanic reactions.

7. Because of the apparent relationship between amorphous constituents and lime susceptibility of this soil, it may be possible to predict the lime stabilization capability of lateritic as well as other types of soil through determination of the amorphous allophanic colloid content.

REFERENCES

1. Alexander, L. T., and Cady, J. C. Genesis and Hardening of Laterite. USDA Tech. Bull. 1282.
2. Bawa, K. S. Laterite Soils and Their Engineering Characteristics. Jour. Soil Mech. and Found. Div., ASCE, Vol. 83, Nov. 1957.
3. Bennett, H. H. Some Comparisons of the Properties of Humid Tropical and Humid-Temperate American Soils, With Special Reference to Indicated Relations Between Chemical Composition and Physical Properties. Soil Science, Vol. 21, 1926, p. 349.
4. Brown, J. W., and Wolfschoon, T. A. Some Chemical and Physical Properties of Representative Soils of the Republic of Panama. 7th Internat. Conf. on Soil Science, Vol. 4, 1960, p. 271.
5. Follett, A. E. C. The Retention of Amorphous Colloidal "Ferric Hydroxide" by Kaolinite. Jour. Soil Science, Vol. 16, 1965, p. 334.
6. Greenland, D. J., and Oades, J. M. Iron Hydroxides and Clay Surfaces. 9th Internat. Conf. on Soil Science, Vol. 1, 1968, p. 657.
7. Hamilton, R. Microscope Studies of Laterite Formation. In Soil Micromorphology, Elsevier Pub. Co., Amsterdam, 1964, p. 269.
8. Hendricks, D. M., and Whitlig, L. D. Andesite Weathering: I. Mineralogical Transformations From Andesite to Saprolite. Jour. Soil Science, Vol. 19, 1968, p. 135.
9. Humbert, R. P. The Genesis of Laterite. Soil Science, Vol. 65, 1948, p. 281.
10. Jackson, M. L. Soil Chemical Analysis. Univ. of Wisc. Press, Madison, 1956.
11. Mackenzie, R. C. The Differential Thermal Investigation of Clays. Mineralogical Society, London, 1957.
12. Maignien, R. Review of Research on Laterites. Nat. Res. Researh IV, UNESCO, Paris, 1966.
13. Mattson, S. Anionic and Cationic Adsorption by Soil Colloidal Materials of Varying $SiO_2/Al_2O_3 + Fe_2O_3$ Ratio. Proc. First Internat. Cong. of Soil Science, Vol. 2, 1927, p. 199.
14. McIntyre, D. S. Effect of Free Ferric Oxide on Soil Structure. Jour. Soil Science, Vol. 7, 1956, p. 302.
15. Mehra, P. O., and Jackson, M. L. Iron Oxide Removal From Soils and Clays by a Dithionite-Citrate System With Sodium Bicarbonate Buffer. Proc. 7th Conf. on Clays and Clay Minerals, Pergamon Press, N. Y., 1960, p. 317.
16. Newill, D. A Laboratory Investigation of Two Red Clays From Kenya. Geotechnique, Vol. 11, 1961, p. 302.

17. Pearring, J. R. A Study of Basic Mineralogical Index Properties of Lateritic Soils, Physical-Chemical and Mineralogical. Tech. Rept. No. AFWL-TR-69-21, USAF Weapons Lab., Kirtland AFB, New Mexico, 1969.
18. Ramashkevich, A. I. Micromorphological Indications of the Processes Associated With the Formation of the Krasnozems (Red-Earths) and the Red-Colored Crust of Weathering in the Transcaucasus. In Soil Micromorphology, Elsevier Pub. Co., Amsterdam, 1964, p. 261.
19. Schmidt-Lorenz, R. Zur Mikromorphologie der Eisen und Aluminumoxydranreicherung Beim Tonmineralablau in Lateriten Kerolas und Ceylons. In Soil Micromorphology, Elsevier Pub. Co., Amsterdam, 1964, p. 297.
20. Sirarajasingham, S., Alexander, L. T., Cady, G., and Cline, M. G. Laterite. Adv. in Agron., Vol. 14, 1962, p. 1.
21. Terzaghi, K. Design and Performance of Sasamua Dam. Proc. Inst. of Civil Eng., Vol. 9, April 1958, p. 369.
22. Thompson, M. R. Lime Reactivity of Illinois Soils. Jour. Soil Mech. and Found. Eng., ASCE, Vol. 92, Sept. 1966.
23. Townsend, F. C., Manke, P. G., and Parcher, J. V. Effects of Remolding on the Properties of a Lateritic Soil. Highway Research Record 284, 1969, pp. 76-84.
24. Winterkorn, H. F., and Chandrasekharan, E. C. Lateritic Soils and Their Stabilization. HRB Bull. 44, 1951, p. 10.
25. Wissa, A. E. Z., and Rurik, H. Chemical Stabilization of Selected Tropical Soils From Panama and Canal Zone. Waterways Experiment Station, WES Rept. No. 3-63, Vicksburg, Miss., 1963.

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